TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOLUME 126, PART III

1961

Edited by the Executive Secretary, under the direction of the Committee on Publications.

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NEW YORK
PUBLISHED BY THE SOCIETY
1961

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FOREWORD

ASCE TRANSACTIONS, 1961, Part III, contains, nominally, all papers published in the Journals of the Irrigation and Drainage, Power, and Sanitary Engineering Divisions, Proceedings of the American Society of Civil Engineers. The following papers were not included in this part because discussion was not complete when the volume was closed:

JOURNAL OF THE IRRIGATION AND DRAINAGE DIVISION

March 1960

Serendipity and the Development of Experimental Meteorology
by Vincent J. Schaefer (Proc. Paper 2393)[b]

Experiments on Treatment of Summer Cumulus Clouds
by Louis J. Battan and A. Richard Kassander, Jr. (Proc. Paper 2394)[b]

Physical Studies of Santa Barbara Storms
by Theodore B. Smith (Proc. Paper 2395)[b]

Seeding of West Coast Winter Storms
by Robert D. Elliott (Proc. Paper 2396)[b]

Seeding of Clouds in Tropical Climates
by Wallace E. Howell (Proc. Paper 2397)[b]

Generator Technology for Cloud Seeding
by D. M. Fuquay (Proc. Paper 2398)[b]

Cloud-Seeding Results in Santa Clara County
by Arnett S. Dennis (Proc. Paper 2399)[b]

Advisory Committee on Weather Control
by Frederic A. Berry (Proc. Paper 2400)[b]

The Nature of Cloud Systems
by Horace R. Byers (Proc. Paper 2401)[b]

Physical Properties of Clouds
by Roscoe R. Braham, Jr. (Proc. Paper 2402)[b]

Evaluation of Seeding Trials
by Arnold Court (Proc. Paper 2403)[b]

Natural Variability of Storm, Seasonal and Annual Precipitation
by Glenn E. Stout (Proc. Paper 2404)[b]

A Weather Modification Program for the Future
by Howard T. Orville (Proc. Paper 2405)[b]

The Santa Barbara Project
by Robin R. Reynolds (Proc. Paper 2406)[b]

June 1960

Irrigation and Drainage Problems in Uruguay
by J. E. Christiansen (Proc. Paper 2494)[a]

The Heritage of Irrigation in Iraq
by M. R. Lewis (Proc. Paper 2495)[a]

September 1960

Methods of Applying Irrigation Water
by Paul H. Berg (Proc. Paper 2595)

Irrigation in Latin America
by Lyman S. Willardson (Proc. Paper 2610)[a]

December 1960

Vortex Tube and Sand Trap
by A. R. Robinson (Proc. Paper 2669)

Los Angeles Water Supply and Irrigation
by Samuel B. Morris (Proc. Paper 2671)

Method for Estimating Consumptive Use of Water for Agriculture
by Wendell C. Munson (Proc. Paper 2672)

Irrigation Systems of the Tigris and Euphrates Valleys
by Stanley S. Butler (Proc. Paper 2673)

Humid Zone Irrigation in Ceylon
by Philip P. Dickinson (Proc. Paper 2682)

JOURNAL OF THE POWER DIVISION

February 1960

Prospecting for Thermal Power Plant Sites
by Reed A. Elliot (Proc. Paper 2361)[a]

Control of Cracking in TVA Concrete Gravity Dams
by Walter F. Emmons, Olav Lavik, and Paul L. Hornby (Proc. Paper 2372)[a]

Additional Aspects of the Enrico Fermi Atomic Power Plant
by N. L. Scott and R. F. Mantey (Proc. Paper 2375)[a]

April 1960

Piratininga Stream-Electric Generating Station
by O. L. Hooper and H. M. Estes (Proc. Paper 2432)

June 1960

Design of Self-Supported Steel Transmission Towers
by R. N. Bergstrom, J. R. Arena, and J. M. Kramer (Proc. Paper 2515)

August 1960

Digital Computers for Trial-Load Analysis of Arch Dams
by L. R. Scrivner (Proc. Paper 2568)

Design of Arch Dams by Trial-Load Method of Analysis
by Merlin D. Copen (Proc. Paper 2569)

Design of Karadji Hydroelectric Project
by Richard D. Harza and Robert F. Edbrooke (Proc. Paper 2579)

JOURNAL OF THE SANITARY ENGINEERING DIVISION

January 1960

Aerobic Metabolism of Potassium Cyanide
by John B. Nesbitt, H. Robert Kohl, and Elmer L. Wagner, Jr. (Proc. Paper 2341)[a]

Role of Price in the Allocation of Water Resources
by Lawrence G. Hines (Proc. Paper 2343)[a]

March 1960

Diffusion in a Sectionally Homogeneous Estuary
by Richard Kent (Proc. Paper 2408)

May 1960

Behavior of Suspensions
by A. W. Bond (Proc. Paper 2474)[a]

Sanitary Engineering Aspects of Nuclear Energy
Progress Report of the Committee on the Sanitary Engineering Division (Proc. Paper 2476)[c]

Design and Cost Considerations in High Rate Sludge Digestion
by Alfred A. Estrada (Proc. Paper 2479)

Waste Treatment at the Shippingport Reactor
by J. R. LaPointe, W. J. Hahn, and E. D. Harward, Jr. (Proc. Paper 2481)

July 1960

Man Versus Environment
Twenty-Seventh Progress Report of the Committee on Sanitary Engineering Research of the Sanitary Engineering Division (Proc. Paper 2549)[c]

Automatic System for Monitoring Water Quality
Twenty-Eighth Progress Report of the Committee on Sanitary Engineering Research of the Sanitary Engineering Division (Proc. Paper 2554)[c]

Solubility of Atmospheric Oxygen in Water
Twenty-Ninth Progress Report of the Committee on Sanitary Engineering Research of the Sanitary Engineering Division (Proc. Paper 2556)[c]

Light Conversion Efficiency of Algae Grown in Sewage
by William J. Oswald (Proc. Paper 2558)

September 1960

Low Pressure Aeration of Water and Sewage
by N. Claes H. Fischerstrōm (Proc. Paper 2607)

November 1960

Investigation of the Corrosive Behavior of Waters
by Werner Stumm (Proc. Paper 2657)

Derivation of Flow Equations for Sewage Sludges
by Vaughn C. Behn (Proc. Paper 2663)

In the foregoing list, the symbol [a] denotes a paper cleared for publication in the Journals prior to December 1, 1959, which will not be published in Transactions. The symbol [b] is used to signify papers that were part of Symposia that were not completed in time to publish the entire group in Transactions. The symbol [c] represents a committee report that will not be included in Transactions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3127

DRAINAGE PROBLEMS OF THE SAN JOAQUIN VALLEY

By William L. Berry,[1] F. ASCE, and Edward D. Stetson,[2] M. ASCE

With Discussion by Messrs. William W. Donnan; and
William L. Berry and Edward D. Stetson

SYNOPSIS

The geography and geology of the San Joaquin Valley of California are outlined herein, with special emphasis on the factors affecting drainage. The existing drainage problems (1959) are unfolded through a description of the cultural changes that have brought them into sharp focus. The objectives and scope of the drainage investigation in progress (1959) are described.

INTRODUCTION

The San Joaquin Valley of California is a vast and rich agricultural area, containing more than 7,000,000 acres of potentially irrigable valley floor lands. Irrigation was initiated in the 1850's and involved some 800,000 acres by 1900. From that point forward development was very rapid, resulting in irrigation of about 2,250,000 acres by 1930, and approximately 4,000,000 acres today. This development has involved many profound changes in the water resources of the valley.

Irrigation engineers have diverted the flow of numerous rivers and have imported water hundreds of miles from others. Large natural lakes have evaporated or have been drained. The normal seasonal distribution of runoff has been altered by detaining flood flows in foothill reservoirs for later release

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Note.—Published essentially as printed here, in September, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2160. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[2] Senior Hydr. Engr., State Dept. of Water Resources, Sacramento, Calif.

during the low flow period of the year. As a result of such developments, often uncoordinated, many serious problems have occurred. It is a credit to the people of the San Joaquin Valley, and to the others who have worked with them, that many of these problems have already been solved. Probably the most perplexing of the remaining unsolved problems are those of drainage.

The particular phase of the drainage problem described herein is one that may be termed the disposal problem. Our current studies of this problem in the San Joaquin Valley include determination of the drainage relationships between various large areas within the valley, determination of requirements for drainage from each of these areas, and the design of facilities to remove the determined quantities of drainage water from the area where the need exists.

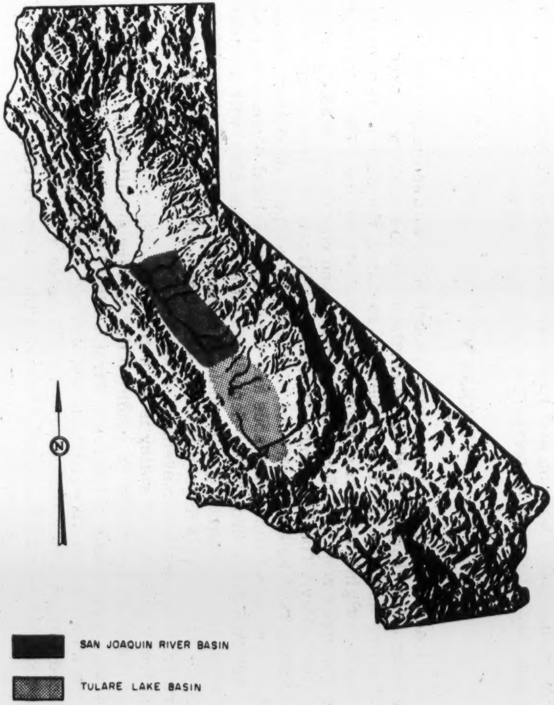
GEOGRAPHY

The San Joaquin Valley, that is drained only by the San Joaquin River, comprises the southern and larger portion of the great Central Valley of California. The floor of the Central Valley extends through $5\frac{1}{2}°$ of latitude, and attains a maximum width in excess of 60 miles. On the north it is bounded by the Cascade Range from which the Sacramento River issues, while the Tehachapi Mountains form the southern boundary. The crest of the massive Sierra Nevada forms the eastern edge of the valley, and the Coast Range separates it from the Pacific Ocean. The northern portion is drained by the Sacramento River and its tributaries and is called the Sacramento Valley. The waters of the Sacramento and San Joaquin Rivers join a delta region, enter Suisun Bay, and then flow into San Francisco Bay through the Carquinez Straits, providing the only natural outlet from the area. The geographic features are shown in a generalized way in Fig. 1.

Referring to Fig. 1, it may be noted that the San Joaquin Valley is composed of two separate basins. The upper basin to the south is known as the Tulare Lake Basin, and the lower or northern portion of the valley is known as the San Joaquin River Basin. Fig. 2 shows in somewhat greater detail the location of the principal streams of the San Joaquin Valley as well as the location of several cities and towns of the area.

Tulare Lake Basin.—For purposes of evaluating water supply and drainage problems, the Tulare Lake Basin should be considered as a closed basin draining into Tulare Lake. It contains approximately one-half of the irrigable valley floor lands of the San Joaquin Valley and receives about one-quarter of the valley's total available runoff. As an indication of water supply conditions, an average annual depth of precipitation of less than 5 in. falls on much of the valley floor in this arid region, but annual runoff from the adjacent mountains is less than 1 acre-ft per acre of irrigable land.

Historical changes in the configuration of Tulare Lake have had considerable bearing on the present drainage problems of the Tulare Lake Basin. At one time the Kings River, in the northern portion of the basin, and probably all other streams that are now tributary to Tulare Lake, were a part of the San Joaquin River system that now drains only the San Joaquin River Basin. However, the reduction in gradient of the Kings River on reaching the valley floor caused deposition of large quantities of material, eventually forming a delta that created a barrier to outflow from the Tulare Lake Basin. Tulare Lake, that was so formed, covered an area of over 850 sq miles at maximum stage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SAN JOAQUIN RIVER BASIN
TULARE LAKE BASIN

FIG. 1.—STATE OF CALIFORNIA MAJOR DRAINAGE SYSTEMS OF SAN JOAQUIN VALLEY

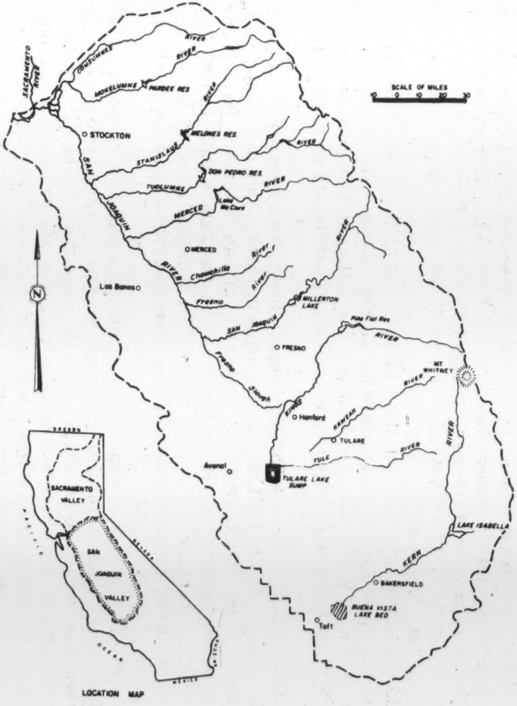
SACRAMENTO RIVER
COSUMNES RIVER
MOKELUMNE RIVER
PARDEE RES
STOCKTON
STANISLAUS RIVER
MELONES RES.
DON PEDRO RES.
TUOLUMNE RIVER
MERCED RIVER
Lake McClure
SAN JOAQUIN RIVER
MERCED
Chowchilla River
Fresno River
Los Banos
MILLERTON LAKE
SAN JOAQUIN
Pine Flat Res
RIVER
FRESNO
Fresno Slough
KINGS
MT WHITNEY
Hanford
KAWEAH RIVER
TULARE
TULE RIVER
RIVER
Avenal
TULARE LAKE SUMP
LAKE ISABELLA
KERN
BAKERSFIELD
BUENA VISTA LAKE BED
Taft
SCALE OF MILES
10 0 10 20 30
N
OREGON
SACRAMENTO VALLEY
SAN JOAQUIN VALLEY
PACIFIC
OCEAN
NEVADA
ARIZONA
MEXICO
LOCATION MAP

FIG. 2.—SAN JOAQUIN RIVER AND TULARE LAKE BASINS

when it spilled to the north. However, the lake has failed to spill for a great number of years, due to high evaporative losses during the long, hot, and dry summers and the extensive diversion of tributary stream flow to higher-lying lands. Construction of flood control dams at the foothill line have caused nearly complete disappearance of the lake. Today (1959) only a few square miles are subject to inundation by uncontrolled winter flows during wetter years, and much of the lake bed is extensively farmed.

A similar situation occurs at the southerly end of the Tulare Lake Basin where Buena Vista Lake was sustained by the Kern River. The history of recession of Buena Vista Lake has followed much the same pattern as Tulare Lake, although the area involved is much less, amounting to only about 40 sq miles.

San Joaquin River Basin.—The San Joaquin River Basin consists of all lands normally drained (as of 1961) by the San Joaquin River. This basin enjoys the advantage of direct drainage to the sea by way of San Francisco Bay. It receives somewhat greater precipitation than does the Tulare Lake Basin. The annual depth of precipitation over the valley floor averages about 12 in., whereas runoff from the tributary mountainous areas averages some 2 acre-ft per irrigable acre.

A geographical feature of significance to drainage problems of the San Joaquin River Basin is the extensive area once subject to annual overflow along the rivers, some portions of which are still subject to occasional overflow. Historically, these lands have been devoted to pasture and quick-growing crops, with irrigation accomplished through natural or intentional flooding only during periods of high flow. Some of these lands are readily reclaimable through ordinary drainage and leaching techniques; however, many have a high concentration of exchangeable sodium, and reclamation requires the addition of soil amendments and other special measures.

GEOLOGY

The San Joaquin Valley is a structural trough filled with sediments to depths varying from a few thousand feet at the northern end to something in excess of 30,000 ft in the southern portion. Generally speaking, the sediments at depth are of marine origin and contain highly saline connate waters. The sediments forming the upper zone are of continental origin and contain the usable ground water of the valley. The continental deposits are largely of river origin, with the discontinuity and heterogeneity associated with this type of deposition. However, there are significant continuous and homogeneous local deposits of lake origin. Additionally, a bed of lake-deposited diatomaceous clay, with a thickness of up to 150 ft, apparently continuously underlies approximately 5,000 sq miles of the valley. This deposit, known as the Corcoran clay, forms an effective barrier to the vertical movement of water.

Thus, the geology of the valley is such that three generally distinct zones of ground water storage exist, a condition that is very significant in the solution of drainage problems of the area. First, at depth we find the connate zone, containing water of sea-like constituency. Second, there is an extensive zone of confined ground water beneath the impermeable Corcoran clay. Finally, there is the free ground water zone overlaying the Corcoran clay. It should be emphasized that this general geologic situation considerably complicates the drainage problems.

IRRIGATION AND DRAINAGE

When irrigated agriculture first began to develop in the San Joaquin Valley, a vast area of higher lying valley lands existed along the east side, fed by apparently adequate streams of high quality water. At the same time a marsh, 5 miles to 50 or more miles wide, occupied the central trough of the valley throughout its length, even giving way to lakes at several points. The area west of the trough in the Tulare Lake Basin was a barren desert for the most part, whereas the west side of the San Joaquin Basin primarily was utilized for stock grazing purposes as west side streams were inadequate as sources of irrigation supply. The major irrigation problem in the valley at that time was one of regulating the extreme seasonal fluctuation of runoff that is so typical of California streams.

Tulare Lake Basin.—Irrigation in the San Joaquin Valley was first practiced adjacent to the Kings River and other east side streams of the Tulare Lake Basin. The developments consisted of diversion of stream flow on a catch-as-catch-can basis with little or no regulatory storage. Water application practices were rather careless, but nevertheless the benefits of irrigation were clearly apparent. Developments based on use of ground water were initiated following appropriation and use of the readily available portion of the surface supply.

In much of the Tulare Lake Basin, ground water was originally obtained by simply drilling a well, as artesian conditions largely prevailed. However, as development continued and water levels were drawn down, the installation of pumps became necessary. Returns from investments in irrigation were so attractive that the ground water was extensively developed, making use of increasingly deeper wells. Today, it is not unusual to find wells 2,500 ft deep with pumping lifts of 400 ft to 600 ft.

In the areas in which the deeper wells have been drilled the major part of the supply is being drawn from beneath the Corcoran clay. Withdrawal from this confined zone far exceeds natural replenishment, and there is generally no possibility for percolation of excess applications of irrigation water. Consequently, a paradoxical situation exists wherein the ground water supply is being exhausted, and yet the danger of buildup of a high water table is very real, having, in fact, occurred in some areas. In some cases the zone above the Corcoran clay can be depended on to yield large enough quantities of water to permit drainage control through pumping. However, this solution of the drainage problem cannot be generally utilized, because the poor mineral quality of the water so obtained may preclude re-use. In any event, such re-use could only be a stop-gap solution, because it would eventually result in intolerable concentrations of minerals in the upper zone. The drainage disposal problem, coupled with the high operating costs associated with ground water pumping, has inspired an unusually high irrigation efficiency in many instances, with little or no percolation through the root zone. This practice is causing higher and higher concentrations of minerals in the upper few feet of soil, and can be expected to seriously impair agricultural productivity in the future, if not corrected.

In the portion of the Tulare Lake Basin that is not generally underlain by a confining clay, the drainage problem is less pressing insofar as presently developed areas are concerned. However, as water is applied to newly developed lands it has the effect of returning to the irrigation cycle all the soluble salts that have been deposited during the centuries that water has been evaporating from the land surface. Although no widespread effect on quality in the ground water basin has as yet been observed, considerable local concern has been expressed because of the increasingly higher mineral concentrations occurring in the return flow, that is generally reapplied lower in the basin.

In reviewing the situation in the Tulare Lake Basin, it becomes clear that there are three basic interrelated factors that affect drainage requirements. The first of these is that more water is presently consumed in the basin than flows into it causing an increased concentration of salts in the ground water basin. A number of projects have been studied to remedy this condition of overdraft, including various proposals for importation of supplemental water from distant sources. The most feasible projects are those that propose the pumped diversion of surplus flows in the delta of the Sacramento and San Joaquin Rivers.

The second basic factor is the excess of salt inflow to the Tulare Lake Basin over the salt outflow. The projects proposed to solve the problem of overdraft will tend to some extent to aggravate this condition. This is true because of the higher mineral concentration in supplemental water to be imported from the delta than is found in natural local supplies in the basin. This problem has been recognized by the state in studies made to evaluate the supplemental requirements of the Tulare Lake Basin. The estimates of water application requirements have been increased to provide for salt balance and in order to permit maintenance of favorable limits to concentration of minerals in the root zone.

The third basic factor affecting drainage requirements in the Tulare Lake Basin, and the one that is probably most significant at the present time, is the leaching action resulting from irrigation of previously undeveloped lands that have been accumulating salts for thousands of years. This will be recognized as a typical drainage problem associated with irrigation reclamation activities in arid regions. The usual solution to such a problem is to provide drainage facilities extending to the nearest major waterway to the sea. In the case of the Tulare Lake Basin the nearest such stream is the San Joaquin River. Unfortunately, the San Joaquin Basin is also experiencing drainage troubles, and the disposal of additional drainage water of poor mineral quality into the San Joaquin River would compound an already difficult problem.

San Joaquin River Basin.—The relation of the development of the water and land resources to the drainage problem in the San Joaquin River Basin is somewhat more complex than in the Tulare Lake Basin. The San Joaquin River flows in a westerly direction to the trough of the valley, then turns sharply northward to constitute the main stream of the valley until it joins the Sacramento River and enters San Francisco Bay. The river is joined at frequent intervals by major tributaries from the Sierra Nevada to the east, but with few and meager tributaries from the arid west side.

Early irrigation development consisted of diversions from the San Joaquin River in the vicinity of the point at which it swings northward. These diversions were utilized primarily for irrigation of adjacent lower-lying lands by wild flooding, in order to increase the production of natural forage crops and hay. Although this development somewhat increased the consumption of water in the basin, it had little effect on over-all drainage conditions.

With the passage of time, however, the advantages of irrigating arable lands of the San Joaquin River Basin became obvious and many irrigation companies and districts were formed. Diversions were made from tributary streams, and not only were the natural summer flows depleted, but regulatory storage was developed in the foothills to permit summer use of a portion of the winter flows. This storage reduced the threat of flooding of the lower lying lands, and thereby encouraged higher uses and additional reclamation in the trough area. Hydroelectric power facilities, involving additional storage, were constructed in the mountainous areas of the basin, with further regulatory effect on the regimen of stream flow.

The rapid growth of the metropolitan area surrounding San Francisco Bay has led to the storage and export of significant quantities of high-quality water from streams of the San Joaquin River Basin. The Central Valley Project has resulted in the export of high quality water from the San Joaquin River to the Tulare Lake Basin, but, more significantly, it has resulted in the importation of water containing greater quantities of dissolved minerals than does water native to the basin.

Ground water extractions within organized districts in the San Joaquin River Basin are generally made primarily for drainage control, although ground water constitutes the principal source of supply for many lands in non-organized areas and for local municipalities. The availability of a relatively plentiful and inexpensive surface water supply in many areas, and the poor quality of the ground water in other areas, have resulted in less extensive development of ground water in the San Joaquin River Basin than has occurred in the Tulare Lake Basin. As a consequence, the basin-wide picture is one of rising ground water levels, as would be expected in an area served primarily from surface sources. However, there are local instances of the lowering of ground water levels associated with overdraft.

Some of the more significant developments that have been described concerning the San Joaquin Basin have occurred in the past decade, and the effects have not been properly evaluated as yet. The basic problem, however, is quite apparent. Progressive depletion of stream flow has occurred due to both beneficial use upstream and export from the basin. This has apparently been accompanied by increased mineral concentrations in accretions of water to the valley floor during the irrigation season, to such an extent that diverters from the main stream are justifiably concerned with protection of the quality of their supply.

PLANNING TO SOLVE THE DRAINAGE PROBLEMS

Recent water project planning for the San Joaquin Valley has taken cognizance of the importance of drainage disposal. The necessity of maintaining root zone salinity within an acceptable range has been acknowledged and has served as a consideration in the evaluation of project water requirements.

The California Water Plan, that has been prepared by the Department of Water Resources as a comprehensive guide to future development and utilization of the water resources of the state, includes a finding that a master drain ultimately will be necessary if the potential productivity of the San Joaquin Valley is to be realized. In June, 1955, several irrigation agencies dependent on the San Joaquin River for their supply entered into an agreement with the state providing for an evaluation of present and future quality of water in the lower reach of the river from which they divert. Finally, the Joint Committee on Water Problems of the California Legislature investigated the drainage problem of the San Joaquin Valley. As a result of recommendations published in this committee's report, the San Joaquin Valley Drainage Investigation was initiated by the California State Department of Water Resources in June of 1957.

The San Joaquin Valley Drainage Investigation, that has been programmed to extend over a 6-yr period, has two broad objectives: (1) Formulation of a detailed master drainage plan for the entire San Joaquin Valley, and (2) Intensive study of areas in present urgent need of drainage, including evaluation of the problems, design of works needed to solve the problems, and all other studies necessary to determine the feasibility of construction of the drainage facilities.

To attain the first objective it is necessary to determine the drainage requirements of all areas of the valley under existing conditions and forecast future conditions of development. To do this, the Dept. of Water Resources is largely utilizing a companion study of theirs to determine the rate of growth of economic demands for supplemental water. This forecast is based on classification of lands, selection of a crop pattern consonant with the land capability, cost of water, and estimates of future market conditions. With this estimated rate of development as a guide, the drainage requirements will then be determined, staged with respect to time for each area. This will enable the department to prepare a detailed physical plan for achieving basin-wide drainage, making use of essentially the same techniques as are used to devise a plan for basin-wide distribution of water.

The second broad objective of this investigation is the development of projects for immediate construction to solve existing urgent problems of drainage disposal. These projects are to be such that they can be integrated to the fullest practicable extent into the master plan. In this case the department is dealing with presently developed areas and can measure many of the factors affecting drainage as well as the damages resulting from its lack.

Determination of the justification for construction of drainage facilities will be accomplished by comparisons of estimated benefits to the area, both with and without project facilities, and also with various proposed facilities in operation. These studies will, of necessity, include consideration of extensive land areas, because the benefit to be derived by providing an outlet for drainage water of poor mineral quality can accrue to lands entirely removed from the area drained.

Finally the financial feasibility of the selected project will be determined. This determination will consist primarily of an exploration of the ability and willingness of all proposed participants in the project to pay their allocated costs.

CONCLUSIONS

The economic demand for water in the San Joaquin Valley is being met by overdraft of the huge reservoir of ground water that underlies the valley. The complex nature of the geology of the valley and the fact that a large part of the area is a closed basin complicates effects of the overdraft, but eventually it can only result in impairment of the quality of the water through concentration of the dissolved minerals to the point where it will no longer be usable.

In the planning and design of water importation projects to correct the overdraft condition, drainage disposal needs must be specifically considered. Otherwise the problem of quality degradation will not be solved, and, in fact, may be aggravated. This is true because the imported water will contain moderate quantities of salt, and will be used to develop large areas of highly salinized land with resultant degradation of the ground and surface water supplies by leaching of soluble minerals from the soil. In many cases, costs of the required disposal facilities should be included in the determination of economic justification and financial feasibility of the project as a whole, because those facilities are as necessary to the continued success of the project as the assurance of a water supply. As attested by historical experience in arid and semiarid areas throughout the world, irrigated land that today is highly productive and capable of supporting a substantial outlay to maintain its productivity can in a few years be reduced to an alkaline waste if proper drainage disposal facilities are not provided.

DISCUSSION

WILLIAM W. DONNAN,[3] F. ASCE.—This paper deals with one of the most important water management problems connected with the entire California Water Plan. The State Department of Water Resources is to be commended for the vigor with which they are attacking the problem.

In order for the comprehensive scheme of water distribution and irrigation of new lands to function properly, an adequate system must be provided for drainage.

Of particular interest is the problem of drainage of the Tulare Lake Basin. This is a two million acre-area of valley floor lands comprising the south portion of the Valley. To quote the authors, "For all practical purposes, the Tulare Lake Basin is a closed basin draining into Tulare Lake." The delta of the Kings River cuts off the lower part of the valley from a natural outlet to the San Francisco Bay. To the average layman this problem seems remote and of little consequence. After all, the water levels in much of this area have a history of gradual recession. With underground supplies being depleted, why fret about drainage problems? Where is the danger?

The danger lies in the overall salt balance. Suppose that careful plans are made to provide just enough water. Enough for full development of agriculture, suburban and industrial potentials. Then, no drainage problem would develop. The thing that would develop would be a gradual buildup of salts in the subsoil and in the underground water supply. Within 50 yr, possibly one-fifth of the land would be affected to some degree with saline elements.

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It seems almost mandatory that some type of drain device or salt balance mechanism be provided for the Tulare Lake Basin. The writer has long been preoccupied with this problem and is fully cognizant of the difficulty inherent in any solution. In fact, as early as 1953, a Master Drain Plan for the Tulare Lake Basin was proposed after a reconnaisance study of the problem.

At the present time three alternative solutions to the problem are recognized.

One solution would be to provide a gravity trunk drain from Buena Vista Lake north to some convenient point in former Tulare Lake bed. Here pumping stations would be built to lift the water about 50 ft to 70 ft up over the barrier created by the Kings River delta.

A second solution, and one which this writer favors, would be to provide a gravity trunk drain from Buena Vista Lake northward for about 55 miles to a point due west of Delano, Calif. At this point, the trunk drain would angle off toward the northeast, roughly on the 215 ft elevation contour bypassing Tulare Lake bed. Then by circling back northwestward to a point one mile west of Stratford, Calif. this main bypass channel would connect with the Kings River wasteway. The bypass channel to this point could be constructed on a grade of about one-half ft per mile. The construction would be such that the western side of the drain channel would consist of a hugh levee.

The bypass channel would, thus, serve two purposes: (1) transport drainage flow originating in the area from Buena Vista Lake northward; and (2) act as a flood control levee to protect Tulare Lake bed from periodic flooding by the Tule, Kaweah, Deer, and other Sierra Nevada streams.

From this junction point, with Kings River wasteway, the drain would be constructed on a direct line northwesterly to Whitesbridge. This alignment would cross the Kings River delta and would require a deep cut of about 37 ft at the deepest point at Summit Lake. The most favorable aspect of this proposal would be an all-gravity system from Buena Vista Lake to the San Joaquin River.

A third solution to the problem would be to collect drainage flow at some convenient point in Tulare Lake bed and desalinize the water. This last proposal would require an economical method of desalinization. There are perhaps other equally meritorious solutions to this problem. More than likely, the final design of drainage will embody the best factors from several alternatives. Whatever the solution may be to this problem, it seems safe to assert that careful and exhaustive investigations are being carried on to determine the most feasible drainage program for the valley.

WILLIAM L. BERRY,[4] F. ASCE, and EDWARD D. STETSON,[5] M. ASCE.—The writers were quite interested in Donnan's discussion of possible alternative physical solutions to the problem of providing drainage disposal to the Tulare Lake Basin of the San Joaquin Valley.

The paper contained no specific solution to the drainage disposal problem because this naturally must await determination of the present and future drainage requirements of each area of the valley, as outlined in the paper. However, considerable progress has been made in the writers' studies since presentation of the paper, and they are now studying various alternative specific solutions.

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[4] Chf. Div. of Resources Planning, Calif. State Dept. of Water Resources.

[5] Senior Hydr. Engr., Div. of Resources Planning, Calif. State Dept. of Water Resources.

An alignment skirting the Tulare Lake Bed on the east, somewhat similar to the one described by Donnan, has been given careful consideration, and may be recommended as a feature for ultimate construction.

The writers agree with Donnan that overall salt balance is important, and this was pointed out in the paper. However, in an area such as the Tulare Lake Basin in which salts have been accumulating for thousands of years, the provision of overall salt balance does not necessarily ensure the continued productivity of segments of the economy dependent on a water supply of a given quality. In our planning, consequently, we are more concerned with concentrations of salts than we are with tons of salt.

Donnan mentions the possibility of desalinization as an alternative to disposal from the basin of the waste water. This is of particular interest in connection with the treatment of wastes originating at some distance from the main waste-production areas. However, even in those circumstances, the cost does not appear to be competitive with hydraulic disposal. It should be noted that after desalinization the salt still requires disposal from the basin.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3128

DRAINAGE AND WATER MANAGEMENT RESEARCH IN WESTERN EUROPE

By William W. Donnan[1], F. ASCE

SYNOPSIS

This paper attempts to indicate to the reader the ease with which an American visitor can communicate with the engineers of western Europe. The author presents a description of the research facilities visited and the names and works of the engineers with whom he conversed and exchanged ideas. In addition, a brief description is presented of the vast number of institutions engaged in this type of research in the Netherlands.

INTRODUCTION

In the late summer of 1958 the author was privileged to make a 3-week study tour of drainage and water management research in Great Britain, Denmark, and the Netherlands. The report of this study tour is written with two objectives in mind, namely: 1) To describe the ideas, tools and techniques of the European engineers and technicians; and 2) To interest the reader enough so that he will inaugurate correspondence and exchange of ideas and information between countries. Toward this end there is listed, under Acknowledgments,

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Note.—Published essentially as printed here, in June, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2527. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Prin. Agric. Engr., Western Soil and Water Mgt. Research Branch, Soil and Water Conservation Research Div., Agric. Research Service, U. S. Dept. of Agric., Pomona, Calif.

a number of names and addresses of individuals who would be valuable first contacts to anyone wishing information on drainage and water management research in the three countries the author visited.

It should be emphasized that contacts with engineers and scientists outside the United States is not difficult. In western Europe the language barrier does not exist. Anyone who is interested in securing more details of the work herein described will find that these contacts are worthwhile and rewarding.

GREAT BRITAIN

The visit to Great Britain centered mainly on the School of Agriculture, University of Cambridge, Cambridge, England. The drainage and soil physics work carried on here is under the direction of E. C. Childs.

Of considerable interest to anyone working in drainage is the famous tank which Childs built in cooperation with the Agricultural Research Council of Great Britain. This tank, estimated to be 50 ft by 50 ft square and 8 ft deep, is used for a variety of experiments on tile drainage, falling water tables and nonsteady flow phenomenon. A feature of this tank experiment is the water application system developed to apply water by means of sprinklers at an infinite variety of volumes. Child's group has developed one device which, it seems, could be incorporated into any of the tanks or laboratory devices where the objective is to determine the free water surface. It is a device for recording pressure distributions in porous media during fluid flow experiments.[2]

Working with Childs on specific problems of research is A. N. Ede, Agricultural Engineer. Ede has done a lot of work developing a machine to lay continuous concrete tile lines behind a mole device, and has published a number of papers on this work.

Recently Ede has been working on field measurements of rainfall, water levels, and tile flow phenomenon in hopes of securing a large backlog of field data. These data along with empirical equations are to be used in developing theories on the nonsteady state conditions of tile drainage. During the conduct of these field measurements of water table and tile flow, Ede and Childs have developed 1) a simple, economical observation well with float recorder and clock in a single unit; and 2) a tile flow recorder which employs the principle of heat transfer, thermistors, and calibrated flow charts for obtaining instantaneous flow measurements.

Childs and Ede have approximately 8 or 10 individual fields where they hope to be able to record rainfall, water level fluctuations, and tile flow. Two interesting field trips were made to the Fenlands Drainage District. These lands were essentially marsh areas until 1880 or thereabouts when the government began extensive drainage works. The drainage system has gradually been improved. At the present time most of the main natural drainage ways, rivers, and streams have been diked up with levee works to entrain their flows to the ocean. Tidal gates and other large control structures have been built. The former marshland areas have been reclaimed with tile drains and are cropped to pasture, grain and forage.

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[2] "The Recording of Pressure Distributions in Porous Media during Fluid Flow Experiments," by T. O'Donnell, D. H. Edwards, N. Collis George, and E. G. Youngs, Journal of the Scientific Instruments, Vol. 35, February, 1958, p. 63.

DENMARK

Denmark has an area of 16,575 sq miles and consists of about 500 islands and the mainland area of Jutland. The country is characterized by an extremely flat topography. One of the highest points in the entire country is Himmelbjerg (Hill) on Jutland, which has an elevation of 565 ft above sea level.

90% of the land area is productive, and nearly 65% is cropped, the balance being in reforestation areas. While one hears quite a lot about the reclamation work of the Netherlands, there is also a great deal of tidal and heath land reclamation work accomplished, and in process in Denmark. Several hundred thousand acres of tidal lands have been reclaimed by building large sea wall dikes to straighten out the coastline and then by draining the newly created, shallow, land-locked areas.

This development work and the work on the heath lands reclamation is carried on by the Danish Heath Society with headquarters at Viborg, Jutland. This agency carries on an intensive program of hydrological measurements somewhat akin to the United States Geological Survey, and it also supervises the drainage engineering program.

Tile drainage in Denmark started in 1850, and today 40% of the arable land is tile drained. The depth of drains ranges from 80 cm to 120 cm. Spacing is predicated upon experience. The soils are extremely heterogeneous. Spacings range from 14 m to 18 m in sand to sandy loam soils. They are usually 10 m in clay soil. Early tile drains were 1 in. in diameter (inside). Today most tiles are 2 in. inside diameter. The trend is toward using 3-in. diameter tiles in the future.

Drainage is more or less subsidized by the Danish government. The large drainage systems, dikes, and pump works are all done by the government. Money is appropriated for these works and they are called reclamation programs. The individual farm drainage systems and tile systems on fields are planned and designed by private engineers. These plans are submitted for review; if approved, the government pays 50% of the total cost. The balance is also paid by the government but is collected from the land owner over a 10-yr period. Tile drainage costs are approximately 2,000 kroner per hectare (800 kroner per acre or $115 per acre). Tile drains are installed by private contractors during the period from September to December. Much of the tiling is still done by hand digging.

Anyone working on the problems of water balance in the soil would profit from contact with H. C. Aysling at the Hydrotechnic Institute, Royal Veterinary College, Copenhagen, Denmark. He is doing some exceptionally interesting work with lysimeters. Aysling has two sets of three lysimeters 2 m-by-2 m square, and made of concrete. These lysimeters are 80 cm deep and their sharp upper lip is 5 cm below ground surface. They have been packed with 15 cm of gravel sand and then filled to ground surface. Water level is held at 45 cm. One set of lysimeters is irrigated; the other set is not irrigated. They are located on a plot of close-cropped grass turf 350 ft square in an agricultural area. At the same location various climatic data are being gathered—wind, rainfall, evaporation, humidity, temperature, radiation, and back radiation. The evaporation pan used is a copy of the Young screen pan. According to Aysling, this pan has proved to be the "integrator" of all those factors which influence the use of moisture by the turf. In fact, since 1957, Aysling has used the pan as his reference for irrigation needs. Whenever the pan needs to be replenished, the irrigated plots are irrigated. The irrigated plots use 10% more water than the nonirrigated plots. All water controls and all automatic measurements are handled from a control bunker built along the south edge of the plot. They are assembling a tremendous amount of data, which is recorded automatically on charts.

Aysling also has tensiometers installed in some plots at depths of 25, 50, 75, 100, 125, and 150 cm. These are of the old type. He has found that roughly half of the available water in the root zone can be used before appreciable reduction in consumptive use and production takes place. This is a moving value for any one crop since as the roots go deeper, the amount of water available becomes greater.

Perhaps the most surprising single revelation of the visit to Denmark was the extreme interest of the Danish people in their water resources. Here is an area which would be classed as essentially humid. Yet everyone seems to be talking about and interested in the most economical use of their water supplies.

There is an obvious conflict of interests, the same as in our arid western United States. Metropolitan Copenhagen is short of water and has drawn down the underground well supply to a point where sea water is intruding in some areas. They would like to create fresh water lakes, and their interests conflict with agrarian groups who would like to drain areas to provide more land. These problems are being worked out gradually through the application of engineering and economics.

NETHERLANDS

The Netherlands is even smaller than Denmark, having only 12,504 sq miles of area. This is about half again as large as Massachusetts but contains over 820 people per sq mile. About half of the area is below sea level. The famous Dutch dikes, sea walls and drainage systems are a requisite to use of much of the land.

The Netherlands is truly a mecca for the drainage engineer, and the main center of activity is at the Agricultural University, Wageningen. This town of about 10,000 to 15,000 is the focal point for a vast complex of institutes and departments dealing with agriculture. Its claim to being the center of agricultural science is well founded.

The following is a partial list of some of the institutes dealing specifically with soil and water management problems. There are perhaps 20 to 30 others dealing with crops, forestry, veterinary medicine, and agricultural industry: 1) State Agricultural University—Department of Land Drainage and Land Improvement; 2) Soil Survey Institute (Bennekom); 3) Laboratory of Soil and Crop Testing (Oosterbeck); 4) Institute for Land and Water Management Research; and 5) International Institute for Land Reclamation and Improvement. In addition, these institutes are closely allied with the previous five listed: 6) Government Service for Land and Water Use (Utrecht) [Comparable to the American Soil Conservation Service, Dept of Agriculture (SCS)]; and 7) Research Department of the Zuiderzee Reclamation Project (Kampen).

Agricultural University Department of Land Drainage and Land Improvement.—Anyone planning to visit the Netherlands to observe their drainage, and soil and water management programs should write to F. Hellinga, at the address indicated in the Acknowledgments.

Hellinga is also Secretary of the Academic Senate. Almost all of the Dutch scientists speak English, thus making it easy to exchange information and ideas. Hellinga's department offers undergraduate and graduate courses in drainage, reclaiming of lands, and relocation. A. J. Zuur also gives graduate courses in reclamation although Zuur's office is in Kampen. D. A. Kraijenhoff van de Leur, a research engineer working with Hellinga, is doing some very interesting theoretical work on hydrology and the falling water table phenomenon.

Soil Survey Institute (Bennekom).—This institute is located at Bennekom, about 2 miles from Wageningen. The director of this institute is F. W. G. Pijls. Pijls has visited the United States.

This group does all the soil mapping and soil classification. They seem to be doing a fine job of soil mapping and are trying to make their mapping more adaptable to the physical planning for engineering design, so that they can be used, for instance, for drainage design purposes. One of the most interesting functions of this institute from a drainage standpoint is their measurements of soil samples. They test for pore space and compile pf curves that are then utilized by the drainage engineer for design purposes. They are also doing pedological research and preparing an overall soils map of the Netherlands on a scale of 1 to 50,000.

Laboratory of Soil and Crop Testing (Oosterbeck).—The director of this laboratory is F. J. A. Dechering.

All the tests on soils and crops are done here on an assembly line basis. They have developed "cook book" methods and also have introduced statistical analysis of the validity of their tests and have created an extremely efficient testing laboratory having a high standard. This laboratory is mentioned because it is felt that such facilities should be considered for our own soil and water research program on a regional basis. In the United States techniques, tools and methods have been developed which, by their very nature, require that the SCS technician, farm advisor, or engineer have available to him a facility for making quick, accurate, diagnostic tests.

Institute for Land and Water Management Research (Wageningen).—The director of this institute is C. van den Berg.

A visit to this institute is an absolute must for anyone working in research on drainage, and soil and water management. The institute was created partly with Marshall Plan money and partly by the Kellogg Foundation. It has adequate offices and laboratories. It is responsible for research in the field of water management (hydrology, irrigation and drainage, plant and soil water relationships), soil improvement and economics of land consolidation. This agency is doing some of the best and most fundamental research on drainage problems of any place visited. It has invested some money in a comprehensive study of rainfall, water table levels, and soils for the Netherlands. The report of this work is now in print.

It is also conducting fundamental theoretical studies of drainage phenomena, particularly the theory of evaporation from a water table and theories on the nonsteady state of drawdown for drainage design. This work is under the supervision of W. C. Visser.

International Institute for Land Reclamation and Improvement.—This institute is located in Wageningen and was founded with the aid of the W. K. Kellogg Foundation. It started its work in 1956. The director is J. M. van Staveren. Van Staveren visited the United States last year.

The institute is responsible for collecting and disseminating information on a world-wide basis with regard to drainage and land reclamation; to promote and conduct applied scientific research on reclamation; and to conduct field investigations world-wide on the feasibility of schemes for agrarian development. In effect, this agency acts as a clearing house for consulting services on agricultural problems throughout the world.

Government Service for Land and Water Use (Utrecht).—This agency in some ways is comparable to our SCS in that it provides technical assistance for farmers on individual soil and water management problems. The man in charge of the Utrecht office is C. L. van Someren.

Van Someren knows more about all types of drainage equipment, machines, tools and techniques than anyone else the author contacted. He has compiled an excellent report on tile-laying machines used throughout the world. He is also familiar with all the various techniques used in the Netherlands to measure field hydraulic conductivity, tile outlet flow, and other applied field techniques.

Research Department of the Zuiderzee Reclamation Project, (Kampen).—The headquarters for this project are at Kampen about 70 miles from Wageningen. The Director of the research, A. J. Zuur, is attached to the University of Wageningen and teaches graduate courses. Zuur is carrying on research on reclaimed soils. The so-called "ripening" of soils which have been drained and reclaimed over a period of years is most interesting and should have some application in the United States.

Reclamation of land from the sea and from tidal marsh areas began about the year 1400 and has continued up to the present time. These projects are long-range programs, and they are carried out segment by segment over a period of years at great cost. When asked how these costs are justified, the author was, in turn, asked, "How much value can you ascribe to productive land with a population density of over 800 people per mile?"

The modern day reclamation of the Zuider Zee area has been in process for approximately 40 yr. This area was once inhabited by Germanic tribes. Newly drained areas reveal the presence of villages and farms from about the year 900. It is thought that catastrophic storms from the North Sea may have inundated large areas in creating the Zuider Zee.

After World War I work was started to close off the Zuider Zee with a sea wall, dike or dam. This work was completed in 1932 and consists of a tremendous dike some 19 miles long and, in places, one-fourth to one-half mile wide. The Ijssel River, a branch of the Rhine delta outlet, flows into this enclosed area forming what is known as Lake Ijssel. The original plan was to carve four huge polders or subdiked areas from this lake. The Wieringermeer Polder of 49,420 acres was put into production in 1935-40. The Northeast Polder of some 118,500 acres was diked off and pumped dry by 1942. Despite delays during World War II, it is now a thriving enterprise. The other two polder areas are in various stages of development with an eventual development of over 500,000 acres.

It must be emphasized that these reclamation works are tremendous in scope and in detail. Not only must the dikes and pump systems be built, but the open drains, tile drains, roads and bridges must also be constructed. Buildings are constructed, trees are planted, parks laid out and people are moved in to take over. The land is leased to a family in perpetuity as long as a male member desires to farm it. These farms are some of the most prosperous and beautiful anywhere in the world.

Obviously all this meticulous planning calls for detailed application of soil and water management. The Dutch are bringing all of their time-honored ingenuity and energy to this problem. Here, too, the industrial and urban competition for water is apparent. One portion of the Zuider Zee project is being revised to provide a large circular fresh water lake around the polders and adjacent to Amsterdam. Even the Netherlands' legendary over-abundance of water is proving to be an asset.

The Netherlands people have even more vast schemes and plan to reclaim tidal lands. They propose eventually to build an outer closing dike project that would encompass all the West Frisian Islands. If they find they have enough fresh water to promote this new project, they may add another million acres of arable land to their inhabitable area.

ACKNOWLEDGMENTS

This paper is a contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, United States Department of Agriculture. Those persons, in foreign countries, who assisted the writer in obtaining the information contained herein are:

Great Britain - E. C. Childs, School of Agriculture, Clare College, Cambridge, England.

Denmark - H. C. Aysling, Hydrotechnic Institute, Royal Veterinary College, Copenhagen, Denmark.

Netherlands - F. Hellinga, Head, Dept. of Land Drainage and Land Improvement, 1 Duivendaal, Wageningen, Netherland.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3130

ACCIDENT PREPAREDNESS IN REACTOR WASTE TREATMENT

By E. D. Harward,[1] A.M. ASCE

SYNOPSIS

Several technical approaches useful in developing emergency procedures in case of an accidental release of liquid radioactive waste from a nuclear reactor are suggested. Some of the difficulties encountered are presented.

INTRODUCTION

It is desirable in nuclear reactor operations to have emergency procedures prepared that could immediately be placed into effect to minimize possible harm to both the general public and to reactor plant operating personnel in case of an accidental release of radioactive material to the environment. Such releases could occur either to the surrounding atmosphere or into an adjoining stream. This paper will discuss some basic considerations that might be incorporated into such an emergency procedure where the accidental release of radioactive material to a stream is involved.

In general, the problem is one of developing a procedure for making an immediate evaluation of an accidental release of radioactivity in terms of the hazard to persons using the stream as a source of water. Such a procedure would not be a substitute for radiation monitoring or a thorough field evaluation of the situation. It is instead, an adjunct to a field evaluation and would serve to determine the future course of a monitoring program. The following

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Note.—Published essentially as printed here, in July, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2543. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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are related as technical considerations in making an evaluation of the radioactive release:

1. Conditions of the radioactive release.
2. Time of flow to downstream water supply intakes.
3. Effect of radioactive decay.
4. Effect of dilution.
5. Emergency radiation limits.

CONDITIONS OF THE RADIOACTIVE RELEASE

In order to develop an emergency procedure for an accidental release of radioactivity to a stream, ways to determine the conditions of the incident must be established and the procedure must be then based on these conditions. They must depend on certain assumptions and on technical data such as those developed in the safeguards reports of the reactor in question. The assumptions must be sufficiently conservative to allow for the unknown aspects of the assumptions. The writer hopes to make clear what some of these "unknowns" might be.

It is necessary to have a means for estimating the quantity of radioactivity released to the stream. This could be accomplished by the use of a radiation monitor and alarm system strategically located on the effluent line leading to the receiving stream. Also of value in making this estimate is the knowledge by the reactor operating personnel as to the type of incident and its potential in terms of a release of radioactivity.

The emergency procedure should be adequate to cover the worst situation as determined from the reactor safeguards report and an analysis of the plant systems involved. One of the most difficult parts of developing a procedure is the job of making conservative, yet realistic, assumptions in postulating the conditions upon which to base the procedure.

TIME OF FLOW TO DOWNSTREAM WATER SUPPLY INTAKES

An important part of the procedure is a method for rapidly estimating the time for the radioactivity that is released to travel to one or more downstream water supply intakes. Such information would be valuable because it would indicate the time available for any action required by the reactor plant operator and local or state health authorities. Under certain conditions it might be desirable to close downstream water intakes temporarily.

The method of estimating downstream flow times suggested here is designed to give only an order of magnitude because of certain limitations. It was developed for the first 100 miles downstream from Shippingport, Pa., and may or may not be applicable to other reactor siles depending on the hydrologic data available. The United States Army Corps of Engineers provided the basic Ohio River data that made it possible to use this method for Shippingport.

Table 1 is an example of the type of data that can be developed for estimating the time of flow. When using the table, it is necessary to know only the total stream flow at the time. At Shippingport, arrangements were made with the Corps of Engineers to obtain certain data from one of their nearby Ohio River dams that would enable a rapid calculation of total river flow by the use of rating curves for that dam. Knowing the total river flow, the time of flow to any of the listed downstream intakes can be read directly from Table 1.

TABLE 1.—TIME OF FLOW IN HOURS DOWNSTREAM FROM REACTOR PLANT TO WATER PLANT INTAKES AT VARIOUS RIVER FLOWS

	Approx. Mileage	6,250 cfs	15,000 cfs	20,000 cfs	25,000 cfs	30,000 cfs	35,000 cfs	40,000 cfs	45,000 cfs	50,000 cfs	55,000 cfs	60,000 cfs	70,000 cfs	80,000 cfs	90,000 cfs	100,000 cfs	125,000 cfs	150,000 cfs	175,000 cfs	200,000 cfs
Reactor Plant	0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Intake No. 1	0.5	2.0	0.6	0.4	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Intake No. 2	8.0	32.0	10	7.3	5.9	5.1	4.4	3.9	3.5	3.2	2.9	2.8	2.6	2.5	2.5	2.4	2.4	2.3	2.2	2.1
Intake No. 3	22.6	90	29	21	17	15	13	12	10	9.5	8.8	8.4	7.7	7.3	7.1	7.0	6.8	6.7	6.5	6.3
Intake No. 4	28.7	115	36	27	22	19	17	15	13	12	11	11	9.8	9.3	8.9	8.8	8.6	8.5	8.3	8.1
Intake No. 5	32.2	129	41	30	25	22	19	17	15	14	13	12	11	10	10	9.9	9.7	9.5	9.3	9.0
Intake No. 6	36.2	145	46	34	28	25	21	19	17	15	14	14	12	12	11	11	11	10	10	10
Intake No. 7	55.4	220	71	53	43	37	32	29	26	24	22	21	19	18	17	17	17	16	16	15
Intake No. 8	59.1	236	75	56	46	39	34	30	27	25	23	22	20	19	19	18	18	17	17	16
Intake No. 9	101.6	406	128	97	78	65	56	50	46	43	41	39	37	36	34	34	32	30	29	27

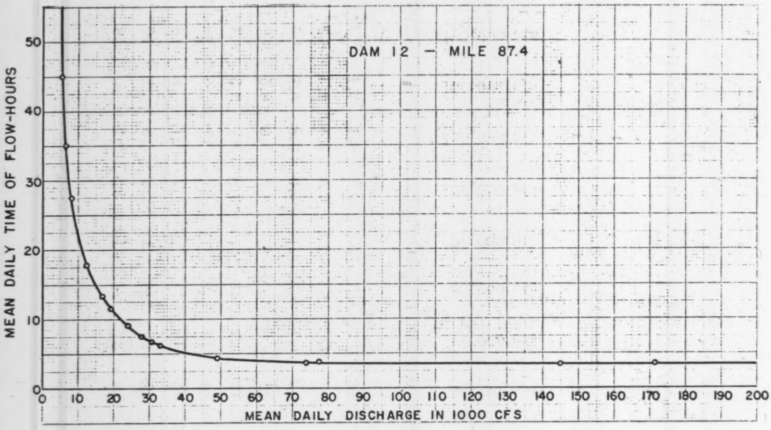
DAM 12 — MILE 87.4
MEAN DAILY TIME OF FLOW-HOURS
0
10
20
30
40
50
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200
MEAN DAILY DISCHARGE IN 1000 CFS

FIG. 1

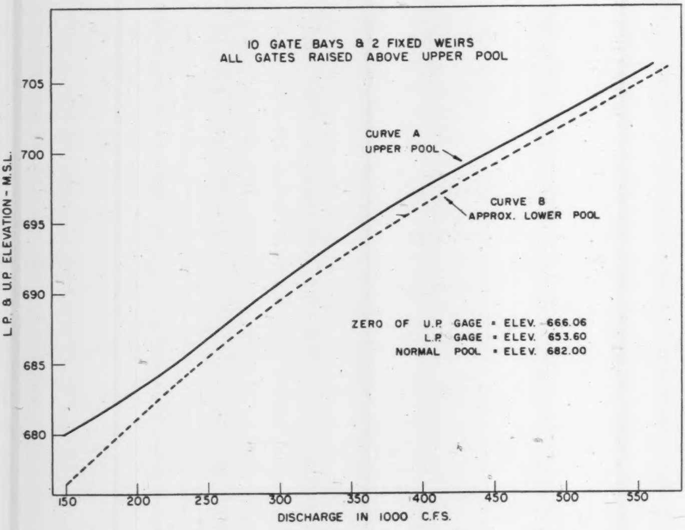
10 GATE BAYS & 2 FIXED WEIRS
ALL GATES RAISED ABOVE UPPER POOL
CURVE A
UPPER POOL
CURVE B
APPROX. LOWER POOL
ZERO OF U.P. GAGE = ELEV. 666.06
L.P. GAGE = ELEV. 653.60
NORMAL POOL = ELEV. 682.00
L.P. & U.P. ELEVATION - M.S.L.
680
685
690
695
700
705
150 200 250 300 350 400 450 500 550
DISCHARGE IN 1000 C.F.S.

FIG. 2

The range of river flows or discharges used in Table 1 was suggested by a monthly-flow duration curve for the Ohio river at Sewickley.[2] The times of flow for various river discharges and distances downstream were computed from a series of mean daily discharge-velocity curves furnished by the Corps of Engineers. An example of one of these curves is shown in Fig. 1. Each curve shows the relationship between river discharge and velocity (or time of flow) for the pool between a pair of Ohio River dams. By obtaining cumulative flow times using a series of curves for a considerable distance, Table 1 can be developed. The flow times to the water supply intakes is merely a distance interpolation between dams.

The total river discharge or flow is determined at any given time by using rating curves for the nearby dam. Such curves are shown in Fig. 2 through 5. Data that are obtained from the dam by telephone are the elevation of both the upper and lower pools (that is, upstream and downstream river elevations), and the number of dam gates opened and the height of their openings. If all gates are raised above the upper poole, Fig. 2 gives you the total river flow directly knowing the upper pool elevation. Fig. 2 gives the total discharge through the gates and over fixed weirs at the dam. If one or more gates are lowered, Fig. 3, 4 and 5 are used to determine river flow knowing upper and lower pool elevations and gate opening information. Fig. 3 is used when the lower pool elevation is below the top of the gate opening while Fig. 4 is used when the lower pool is at or above the top of the gate opening (submerged flow). Curve A of Fig. 5 is used to compute the discharge through one open gate with gate above the upper pool. Curve B of Fig. 5 determines the flow through the fixed weirs. The combined discharges for each of the gates and the fixed weirs is the total river flow. A computation sheet for quickly making the estimate can be developed. An example of such a sheet is shown in Table 2. It will be noted in Table 2 that certain elevation numbers used in the computation are fixed.

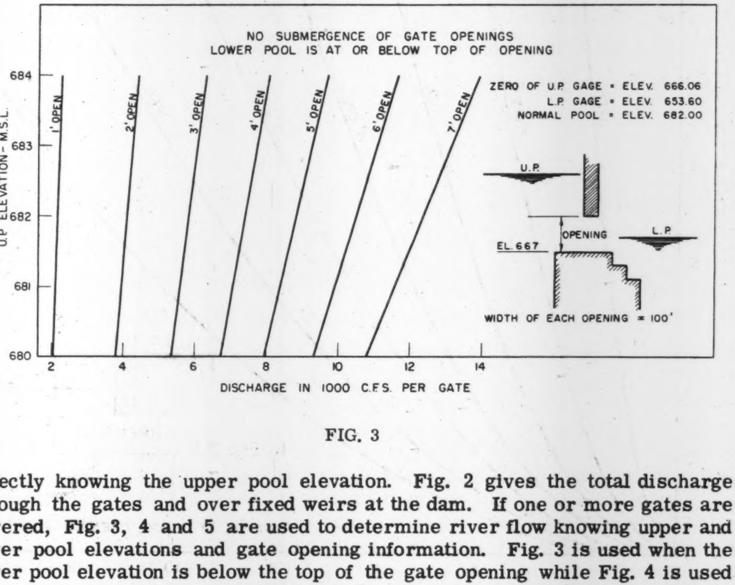

FIG. 3

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[2] "Water Quality and Flow Variations in the Ohio River 1951-55," Ohio River Valley Water Sanitation Comm., Cincinnati, Ohio, 1957, p. 17.

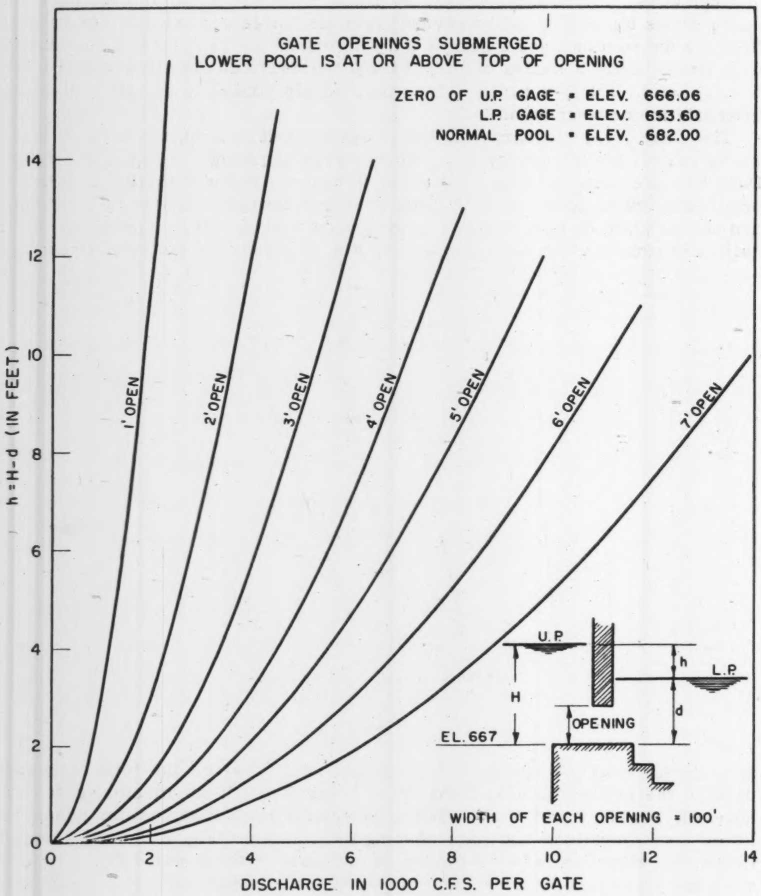
GATE OPENINGS SUBMERGED
LOWER POOL IS AT OR ABOVE TOP OF OPENING
ZERO OF U.P. GAGE = ELEV. 666.06
L.P. GAGE = ELEV. 653.60
NORMAL POOL = ELEV. 682.00
h = H-d (IN FEET)
0
2
4
6
8
10
12
14
1' OPEN
2' OPEN
3' OPEN
4' OPEN
5' OPEN
6' OPEN
7' OPEN
U.P.
L.P.
h
H
d
OPENING
EL. 667
WIDTH OF EACH OPENING = 100'
DISCHARGE IN 1000 C.F.S. PER GATE

FIG. 4

EFFECT OF RADIOACTIVE DECAY

The fission products released to the stream undergo radioactive decay during their travel downstream. The amount of this decay can be estimated for the various flow times to downstream points by using a curve showing the relationship between time and gross decay factor. The gross decay factor is merely the ratio of initial radioactivity to that activity at anytime. An example of such a curve is shown in Fig. 6. Knowing the estimated radioactivity released and the estimated time of flow to downstream intakes, one may readily determine the decay factor from the curve and estimate the reduction in radioactivity due to decay by the expression

$$\text{Downstream Activity} = \frac{\text{Estimated Activity Released}}{\text{Gross Decay Factor}}$$

Although the mechanism for making the estimate is simple, the work in developing the gross decay factor curve can be quite involved. The best way to calculate the curve is to sum the activities of all the individual fission products of concern at various times at and following reactor shutdown. The ratios of the gross fission product activity in the reactor core at shutdown (t = 0) to gross fission product activity at various decay times (t) are plotted against the corresponding decay times. This type of curve is very flexible since the decay factors for fission products released at any interval of time after reactor shutdown can be obtained.

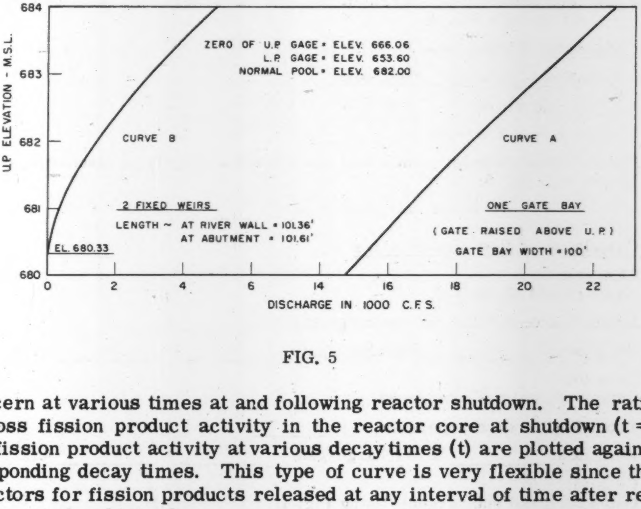

FIG. 5

In computing the gross activities by making a summation of activities in the reactor core at shutdown and at time intervals afterwards, all fission products need not be considered. Some fission products will not leave the plant due to their deposition on pipe walls, components, and so on. Fission products that exhibit this tendency have been reported in the literature.[3] For conservatism, the gross fission product activity can be calculated for reactor shutdown after some prolonged period of continuous full power reactor operation. All computations for the Shippingport plant are based on 3,000 continuous full power

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[3] "Effects of Irradiation on Bulk UO2," by J. D. Eichenberg, P. W. Frank, T. J. Kissel, B. Lustman, and K. H. Vogel, WAPD-183, October, 1957. (Available from Office of Tech. Services, Dept. of Commerce, Washington, D. C.)

TABLE 2.—EXAMPLE OF COMPUTATION SHEET FOR ESTIMATE OF DOWNSTREAM TIME OF FLOW

A. **Basic Data Obtained from Dam**

Number of gates open __________

Height of each gate opening __________

Upper Pool (U.P.) Elevation __________

Lower Pool (L.P.) Elevation __________

Determine Gate Submergence:

Gate opening flowline elevation = 667.0

Gate Opening = __________

Elevation of Gate =

Case (a): For those gates having elevations greater than U.P. elevation, use Curve A of Fig. 5.

Case (b): For gates having elevation greater than L.P. elevation, use non-submerged conditions with Fig. 3.

Case (c): For gates having elevation less than L.P. elevation, use submerged condition with Fig. 4.

B. **Dam Discharge Computation**

1. **Calculation of flow over fixed weirs**

Weir Elevation = 680.33

Upper Pool Elevation = __________

If U.P. elevation greater than weir elevation thus signifying discharge, from "Curve B," Fig. 5:

Fixed weir discharge = __________ cfs

2. **Calculation of flow through gates**

Case (a):
Gates raised above U.P.

From "Curve A," Fig. 5; Discharge/gate = __________

No. gates __________ x discharge/gate __________ = __________ cfs

Case (b):
Gates open but not submerged

From Fig. 3; Discharge/gate = __________

No. gates __________ x discharge/gate __________ = __________ cfs

Case (c):
Gates open and submerged; Using Fig. 4:

H = U.P. elevation __________ = 667.0 = __________

d = L.P. elevation __________ = 667.0 = __________

h = H d = __________ __________ = __________

From the curve, discharge/gate = __________

No. gates __________ x discharge/gate __________ = __________ cfs

Total River Discharge = __________ cfs

C. **Time of Flow Computation**

Using Total River Discharge and Table 1:

Intake	Estimated Time from Reactor Plant

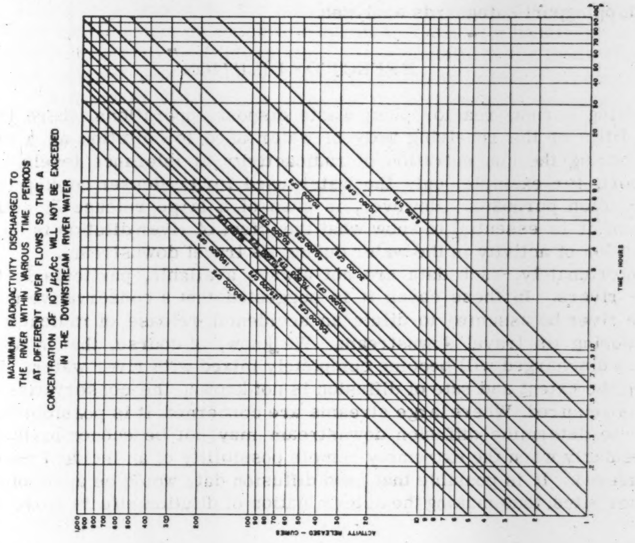

FIG. 7

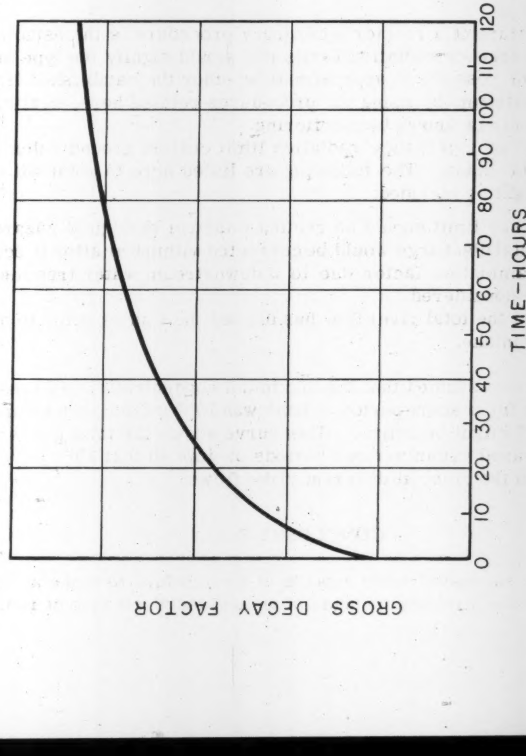

FIG. 6.—GROSS DECAY FACTORS VERSUS TIME AFTER REACTOR SHUTDOWN

hours of operation prior to shutdown, which is consistent with the basis for the Shippingport safeguards analyses.

EFFECT OF DILUTION

During normal reactor plant waste disposal operations, there is usually very little of the receiving body of water used for dilution as a means for maintaining the concentration of radioactivity at tolerance levels. At Shippingport, for example, only the water used for condenser cooling is utilized for dilution purposes. However, in case of a large release of activity to a stream, it is essential to know what the effect of river dilution is on the concentration of activity in the water during its travel downstream.

Unfortunately, such data are not always available, particularly for very large rivers. In these cases it is suggested that a conservative percentage of the river be assumed to dilute an accidental release of radioactive material during its travel downstream. We know, of course, that in time, liquid wastes discharged will become completely mixed with river water. However, where the extent and rate of diffusion is not known, the conservative assumption is required. Where large streams are concerned, it is possible that a program to determine diffusion downstream may not be economically feasible, particularly when the extremely remote possibility of an incident occurring is considered. It is possible that good diffusion data would be available at some reactor sites, thus making the determination of dilution effects more realistic.

EMERGENCY LIMITS

A desirable feature of a reactor emergency procedure is the establishment of appropriate emergency radiation limits that would signify the type of action required. It may be possible to approximate whether the established limit will be exceeded downstream by using the procedures related here, realizing that all the facts can only be known by monitoring.

In establishing an emergency radiation limit certain ground rules and assumptions must be made. The following are listed here to point out some of the things that might be included:

1. The emergency limit should be related to actual biological hazard.
2. Any accidental discharge would be corrected within 6 hr after it occurred.
3. Any decontamination factor due to a downstream water treatment process would not be considered.
4. 10% to 25% of the total river flow has diluted the wastes prior to arriving at a water supply intake.

For example, if one assumed that the maximum concentration of gross radioactivity allowable for a short period of time was 10^{-5} μc/cc, then a curve such as shown in Fig. 7 might be helpful. This curve shows the total gross activity that could be released within various periods of time so that 10^{-5} μc/cc would not be exceeded in the river at different river flows.

CONCLUSIONS

This paper has suggested some aspects of a procedure to make an immediate evaluation of the problem in case of an accidental release of radioactive material to a stream. Undoubtedly, other approaches are possible and improvements could be made on the considerations presented here. A lack of good data in certain areas on which to base an analysis is a handicap. An example might be river dilution information. It is doubtful if the funds required to study the desired river dilution characteristics of a large river could be completely justified in view of the many safety devices and conservative design requirements incorporated in a reactor plant such as Shippingport. Even though the probability of an incident involving the release of large quantities of radioactivity is extremely small, it is considered to be only good judgment to have a procedure to handle such unforeseen events.

It should also be reemphasized that an emergency procedure is only as good as its assumptions and technical data on which it is based.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3131

CONTAINMENT STUDIES FOR AN ATOMIC POWER PLANT

By William McGuire,[1] F. ASCE, and Gordon P. Fisher,[2] F. ASCE

SYNOPSIS

A summary is given of the structural aspects of insuring complete containment for the Enrico Fermi Atomic Power Plant in the improbable event of an accident. The nature of the critical loading, as well as methods of investigation of structural behavior as a consequence of the loading, is discussed for such emergency conditions.

INTRODUCTION

The Enrico Fermi Atomic Power Plant is under construction (1959) on the shore of Lake Erie about 30 miles southwest of Detroit and about 6 miles east of Monroe, Mich.

In all stages of planning and design, safety of operation has been a primary criterion. All possible sources of energy-releasing accidents have been exhaustively studied and positive preventive measures have been taken which reduce the likelihood of their occurrence to essentially zero. However, the designers have gone a step further by assuming simultaneous or sequential failure of several safety devices resulting in the rapid release of thermal or nuclear energy, and by requiring that the products of such accidents be absolutely contained within the reactor building. Thus, the primary concern of containment analysis is not with the performance of the structure during normal operation, but with its behavior under these emergency conditions. This paper is a sum-

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Note.—Published essentially as printed here, in June, 1960, in the Journal of the Power Division, as Proceedings Paper 2508. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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mary of some of the containment problems, the methods used to investigate them, and the steps taken to ensure the integrity of the structure in the highly improbable event of an accident.

Although all of the atomic power plants that have been erected in this country in the past decade have been designed to meet exacting standards of reliability and safety, there has not been formulated, as yet, any general method or approach for studying containment problems. This has led inevitably to the overdesign of containment structures in the interests of safety. It is hoped that as more evidence on the nature of nuclear excursions and structural response to dynamic loads is accumulated, more realistic and economical design procedures can be developed.

DESCRIPTION OF THE FACILITY

A brief description of the reactor, shielding, and containment features is necessary for an understanding of the containment problems. Complete descriptions of the plant are available elsewhere (2,3,4).[3]

Fig. 1 is a sectional elevation of the reactor plant. Liquid sodium is pumped into the bottom of the reactor, flows upward through the core and blanket, and is carried off to an intermediate heat exchanger from which it returns to the sodium pump. There are three identical closed loops of this type. Secondary sodium loops conduct heat from the heat exchangers to steam generators located outside the containment vessel. Power is generated in a conventional turbo-generator of 150,000 kw rated capacity located in a separate building.

The core of the reactor, containing U-235 and U-238 fuel alloy elements and control rods, approximates a cylinder 31 in. in diameter and height. Surrounding this are blanket elements containing U-238 alloy. These, in turn, are surrounded by approximately 12 in. of stainless steel in the form of removable rods, laminated plates, and the lower reactor vessel wall. The main function of the stainless steel is to serve as a thermal and radiation shield.

The reactor vessel is divided into three compartments: the lower reactor vessel, the upper reactor vessel, and the transfer rotor container (Fig. 1). The stainless steel upper vessel, which contains fuel handling, positioning and control equipment, is about 14 ft in diameter and 1-1/2 in. thick and lined with 3-1/2 in. of stainless steel plate. At its top is a circular rotating plug about 12 ft high and about 9 ft in diameter. It is made of laminations of stainless steel, borated graphite, cast boron steel, carbon steel, and stainless steel wool, and weighs about 110 tons bare or 140 tons with all accessories. The plug rests on a ring of ball bearings on the reactor walls but is not anchored to the vessel. The transfer rotor container, which is the transfer area between the external and internal fuel handling systems, is offset from the lower reactor vessel. The entire reactor vessel is supported at the transition between the upper and lower vessels by steel struts.

Surrounding the reactor vessel is the primary shield tank, a 5/8-in. thick carbon steel cylinder with a maximum diameter of 24 ft. It is surmounted by a dome about 18 ft in diameter and 15 ft high. All the space between the primary shield tank and the reactor vessel not occupied by structures or piping, is filled with clay brick, borated graphite, ordinary graphite or carbon, with an average density of 100 lb per cu ft. The total weight of this shielding material is approximately 800,000 lb.

---

[3] Numerals in parentheses refer to corresponding items in the Appendix.

The secondary shield wall (Figs. 1, 2, 3) is composed of intrusion, prepacked concrete with a minimum strength of 3,000 psi. Each face of the wall is completely covered with 1/2-in. carbon steel plate, fully butt welded at all joints. This serves as formwork for the concrete and is firmly anchored to it by transverse tie bars spaced 21 in. on center in each direction. The thickness of concrete varies from 31 in. to 39 in. The effective height of the wall is about 31 ft.

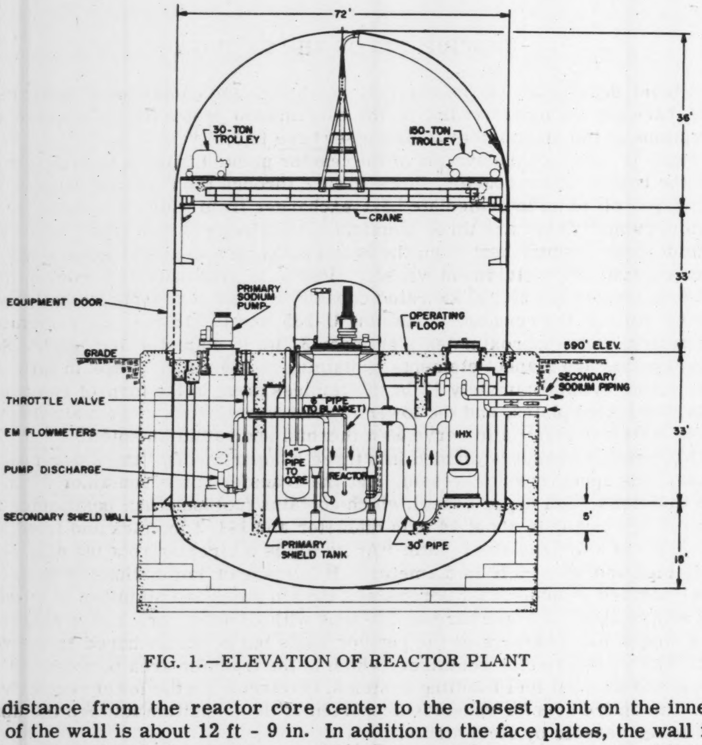

72'
30-TON TROLLEY
150-TON TROLLEY
CRANE
36'
33'
EQUIPMENT DOOR
PRIMARY SODIUM PUMP
OPERATING FLOOR
GRADE
590' ELEV.
SECONDARY SODIUM PIPING
THROTTLE VALVE
8" PIPE (TO BLANKET)
EM FLOWMETERS
IHX
33'
14" PIPE TO CORE
REACTOR
PUMP DISCHARGE
5'
SECONDARY SHIELD WALL
PRIMARY SHIELD TANK
30" PIPE
18'

FIG. 1.—ELEVATION OF REACTOR PLANT

The distance from the reactor core center to the closest point on the inner face of the wall is about 12 ft - 9 in. In addition to the face plates, the wall is reinforced vertically by two layers of No. 7 bars at 6 in. on centers and horizontally by two layers of No. 6 bars at 12 in. on center. Special reinforcement is provided at all openings. There are also thirteen 14 WF 167 columns embedded in the wall for the support of the operating floor above.

The operating floor, also heavily reinforced and about 5 ft thick, is primarily a biological shield. The top 1/4-in. is of steel plate. On the bottom there is a minimum of 3-1/2 in. of steel plate. The floor is supported by a network of steel beams spanning between the columns embedded in the secondary shield wall and an outer row of seventeen 14 WF 167 columns.

The containment vessel is a vertical cylinder 72 ft in diameter with a hemispherical head and hemiellipsoidal bottom. Its overall height is 120 ft. It is of all-welded construction using the American Society of Testing Materials (ASTM) A - 201 B steel furnished to meet the requirements of ASTM A - 300, which provides for a minimum Charpy impact resistance of 15 ft-lb at -50° F. Plates are normalized and aluminum-killed. Design is in accordance with the American Society of Mechanical Engineers (ASME) Code for Unfired Pressure Vessels, Section VIII, and the vessel has been so stamped. Full penetration butt welds are used in the plate. The welds have been radiographed and the entire vessel has been subjected to a 40 psi pressure test as required by the ASME Code. Most of the plates in the cylinder and bottom are 1.03 in. thick and 0.52 in. in the dome. The vessel was designed conventionally as a pressure vessel for 32 psig internal pressure and for 2 psig external pressure plus normal wind and snow loads. It was also designed to support a 150 ton crane riding on circular girders welded to the shell. The lower hemiellipsoidal section is surrounded inside and out by concrete which provides continuous support down to the underlying rock.

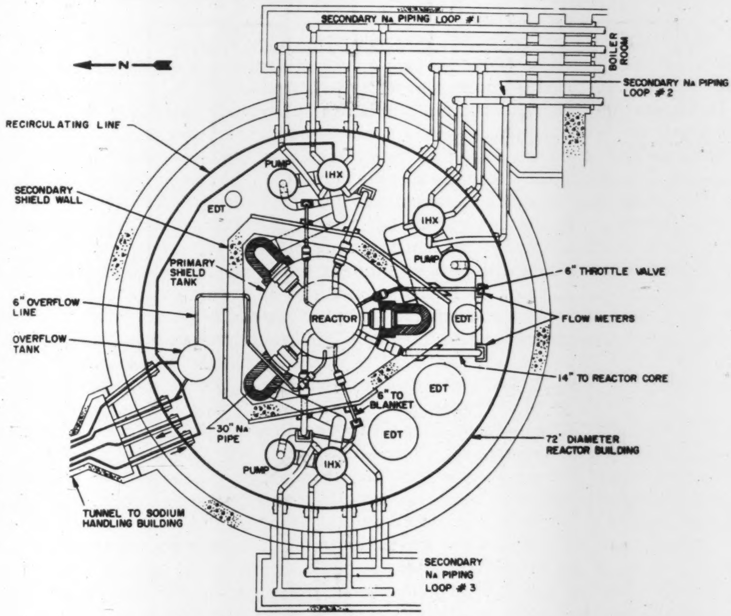
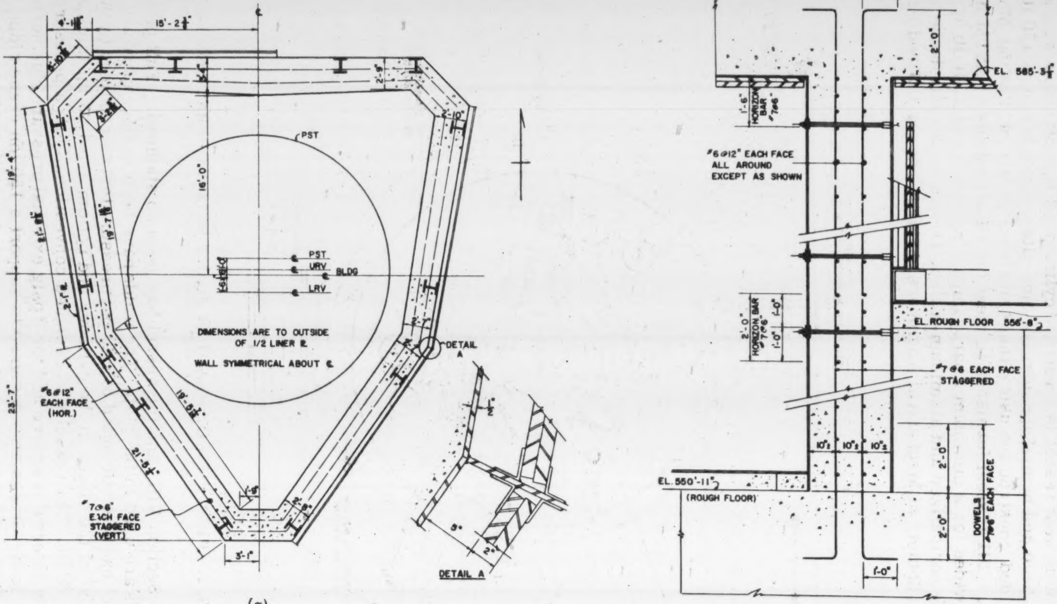

SECONDARY Na PIPING LOOP # 1
N
BOILER ROOM
SECONDARY Na PIPING LOOP # 2
RECIRCULATING LINE
PUMP
IHX
SECONDARY SHIELD WALL
EDT
IHX
PUMP
PRIMARY SHIELD TANK
6" THROTTLE VALVE
6" OVERFLOW LINE
REACTOR
EDT
FLOW METERS
OVERFLOW TANK
14" TO REACTOR CORE
EDT
6" TO BLANKET
30" Na PIPE
EDT
72' DIAMETER REACTOR BUILDING
PUMP
IHX
TUNNEL TO SODIUM HANDLING BUILDING
SECONDARY Na PIPING LOOP # 3

FIG. 2.—PLAN VIEW OF REACTOR PLANT BELOW OPERATING FLOOR


4'-1 1/16"
15'-2 1/4"
℄
5'-10 1/2"
3'-4"
2'-9"
2'-10"
3'-2 1/8"
PST
16'-0"
19'-4"
21'-8 1/2"
19'-7 1/8"
PST
URV
BLDG
LRV
2'-1" ℞
DIMENSIONS ARE TO OUTSIDE OF 1/2 LINER ℞
WALL SYMMETRICAL ABOUT ℄
2'-7"
DETAIL A
23'-7"
#6 @ 12" EACH FACE (HOR.)
19'-5 1/2"
21'-5 1/4"
1'-5"
2'-8"
#7 @ 6" EACH FACE STAGGERED (VERT.)
3'-1"
1/2"
5"
2"
DETAIL A
1'-0"
2'-0"
EL. 585'-3 1/2"
1'-6"
HORIZON BAR #7 @ 6"
#6 @ 12" EACH FACE ALL AROUND EXCEPT AS SHOWN
2'-0"
HORIZON BAR #7 @ 6"
1'-0"
1'-0"
EL. ROUGH FLOOR 556'-8"
#7 @ 6 EACH FACE STAGGERED
10"±
10"±
10"±
EL. 550'-11"
(ROUGH FLOOR)
2'-0"
2'-0"
DOWELS #7 @ 6" EACH FACE
1'-0"

(*a*) (*b*)

FIG. 3.—SECTIONS THROUGH SECONDARY SHIELD WALL

There is a 13 ft - 6 in. high by 11 ft - 3 in. wide equipment door in the shell above the operating floor, as well as two air locks, one 3 ft by 6 ft, the other 3 ft in diameter. There are numerous other small penetrations above and below the floor.

In addition to the stiffening provided by the crane girders, there is a lower internal stiffening ring just below the operating floor. Surrounding the containment vessel below the operating floor level is a plain concrete biological shield with a minimum thickness of 3 ft - 0 in. This is isolated from the vessel by shielded joints.

NATURE AND MAGNITUDE OF ASSUMED EMERGENCY LOADS

Two types of possible accidents were postulated for the Fermi plant by reactor physicists: a sodium fire and a nuclear excursion. Extensive calculations and tests by the Atomic Power Development Assoc. Inc. (APDA) have shown that a sodium fire would result in a maximum possible pressure of 28 psig within the containment vessel and a shell temperature of not more than 460° F. Furthermore, this temperature would not be reached until about 24 hr after the initiating incident. This presents a conventional problem in pressure vessel design. The hypothetical nuclear excursion, although the result of atomic fission, could not remotely approach the energy release of an atomic weapon because there is no way to maintain a critical configuration of fuel long enough to cause this to occur, nor is the assembly rate sufficiently rapid. In fact, any accidental energy release of this kind is based on a set of highly improbable premises that make the possibility of its occurrence extremely remote. Within the limits of such assumptions, APDA physicists have estimated that the maximum credible energy release would be less than that of 1,000 lb of TNT, or if more reasonably based, of the order of 300 lb of TNT. It was also determined that the rate of energy release from an excursion would be of the same order of magnitude as that from ordinary high explosives such as TNT or Pentolite.

Since excursions of this order would cause irreparable damage to the reactor, it is not necessary to keep stresses in the supporting and surrounding structure below the conventional allowable values. Rather, the problem is to investigate all possible modes of failure and to make sure that either the ultimate capacity of each part of the structure is not reached, or that any internal structural failure is not serious enough to cause breaching of the outer containment vessel. Thus, the analytical approach is logically one of seeking reasonable upper limits of loading on each structural element and making sure that the lower limit of the element's resistance to this loading is not exceeded. The major considerations in containment analysis, wherein lie the most serious difficulties of decision, are 1) the methods from structural mechanics that most reasonably define structural behavior, and 2) the nature and magnitude of the loading. In the first instance, substantial theoretical and experimental evidence on the behavior of statically loaded structures near failure has been accumulated only in the last twenty years (since 1940) and reliable analytical procedures are still in the formative stages. Even less quantitative information is available on the failure of structures subjected to dynamic loads. Obviously then, no precise analysis is possible. But, since absolute integrity against rupture of the outer containment vessel must be guaranteed, there is little choice but to use the information and methods presently available and to make conservative estimates of the structural action. In the second instance, there are no known available empirical data on the blast pressures and impulses resulting from nuclear excursions, so that data for conventional high explosives are resorted to. The blast effects of high explosives in free air are well-established. On the other hand, within a geometrically complex structure containing solid and liquid materials of various energy absorbing characteristics, blast effects are more difficult to predict.

The response of a structure to blast loading can be predicted only if the pressure-time history of the impulse is known. In particular, the air blast on any affected structural element may be defined in terms of the maximum instantaneous overpressure, the duration of the impulse, and the shape of the pressure-time curve. These characteristics and associated scaling laws, which permit their application to structures located at varying distances from the blast center, have been developed and corroborated for high explosives. An important question to be settled before using these data for excursion simulation is whether the rate of energy release is of the same order of magnitude for the high explosive as for the nuclear excursion it is intended to represent. Complete agreement between power-time curves cannot be expected, nor is it necessary for estimating ultimate containment capacity. The essence of the comparison is whether the energy release occurs in a matter of microseconds for one and milliseconds for the other. The critical rate of power increase for high explosives is always very rapid and the majority of the energy is released in microseconds, being largely independent of the quantity of explosive (5). For the postulated nuclear excursions, the major fraction of the energy would be released in about 60 microseconds. This correspondence justifies the use of conventional blast data for the Fermi reactor. In this sense, high explosive data may not be appropriate for use with other reactors.

The best available information on air blast about high explosive charges is considered to be that reported by Hoffman and Mills (6) which gives explicit data on peak pressures, impulses and scaling laws. Briefly, the scaling law that permits their application in containment studies states that a given pressure will occur at a distance from an explosion proportional to the cube root of the energy yield, or specifically, that identical peak pressures will be experienced at the same scaled distance, $Z = R/W^{1/3}$. Thus, general curves of pressure and impulse plotted against scaled distance may be determined from test explosions, and then used to find the pressure and impulse for any other charge-weight and blast-radius. Cube root scaling may be applied with confidence over a wide range of explosion energies (7). Scaling applies equally well to all blast parameters such as maximum pressure, duration of positive pressure phase, time of arrival of shock front, scaled impulse, and so forth.

The maximum pressure increase over ambient pressure experienced by a solid wall as it is struck by a shock wave traveling on a path perpendicular to a wall is known as the peak face-on (or reflected) over-pressure. It may be from two to about eight times the corresponding side-on overpressure, which is the overpressure behind a shock wave of the same strength, but unimpeded by a reflecting surface. As a shock wave reflects from a wall, the face-on overpressure decays rapidly from the peak value as in Fig. 4(a). The positive pressure phase is usually followed by a negative pressure or rarefaction phase of much smaller magnitude.

The pressure decay is so nearly linear and the rise time to peak pressure so nearly instantaneous that it is customary to assume a simple pressure-time relationship, $p = p_o\left(1 - \frac{t}{t_1}\right)$, and zero rise time, as in Fig. 4(b). The small rarefaction phase is generally neglected. This approximate linear relationship accords well with experimental data available and has the further virtue of being simple mathematically, resulting in response expressions in closed form.

For explosions confined within closed vessels, there exists the possibility that the initial shock wave may reflect from the vessel walls, converge back on the blast center to form another strong shock which then would rediverge

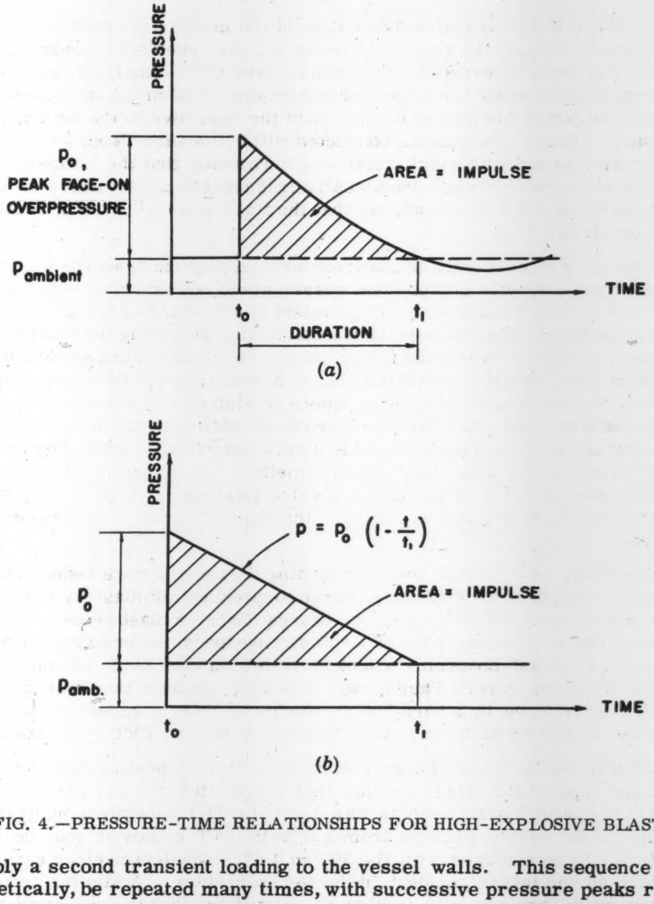


FIG. 4.—PRESSURE-TIME RELATIONSHIPS FOR HIGH-EXPLOSIVE BLAST

to apply a second transient loading to the vessel walls. This sequence may, theoretically, be repeated many times, with successive pressure peaks reducing in magnitude as kinetic energy is converted to other forms. This phenomenon has been observed experimentally by several investigators (5,6,8). However, the possibility of damage from this cause may be discounted for this reactor study on the basis that secondary wave reflections subsequent to the initial shock, if they occur at all, will be much smaller than the initial peak overpressure and negligible in effect. It is highly unlikely that a reactor core excursion, within several heavy and geometrically irregular containers and shields, could produce secondary shocks in the face of the energy absorbed by such internals and the inability to refocus reflected waves.

An excellent summation of the secondary shock problem has been made by W. E. Baker (9) in which he describes theoretical and experimental studies on shells subjected to internal blast loading. In part, he says:

> "Since there is considerable dissipation of the mechanical energy of the blast wave as heat, the repeated shocks will be repeatedly weaker and weaker, until no discernible shock can be detected. The final result of the process is a static pressure and temperature rise in the shell caused by the addition of the heat of detonation of the explosive to the air within the shell. Some experiments conducted within the (Aberdeen, Md.) 30-ft diameter spherical blast chamber . . . have shown that the initial blast loading of a spherical shell from centrally located explosive charges far exceeds the secondary shocks, and that the static pressure rise is usually insignificant."

The main structural units of the reactor building, such as the secondary shield wall and the outer containment vessel, would not experience the full explosive effects of a postulated TNT equivalent excursion. Energy would be expended in crushing the reactor blanket material and rupturing the reactor vessel, thus reducing that available for producing air blast. Further attenuation of the blast effect would result from energy lost in irreversible conversion to heat, crushing and fragmenting the graphite shielding, and heating and vaporizing the sodium coolant. The problem of estimating such energy losses is under very active investigation but it is a very complicated one. The authors know of no currently available, reliable method for computing these losses with certainty. However, it is certain that the total energy loss is very large. In another study of the Fermi reactor, the Naval Ordnance Laboratory has stated (10):

> ". . . a heavy case such as that surrounding the reactor core reduces the effective weight of an explosive charge to produce air blast by at least 70% compared to a similar uncased charge. Greater blast reduction exceeding that of steel-cased high-explosive weapons can be expected because of the blast absorbing potential of the various kinds of material surrounding the core. Thus an air shock not greater than that which would be produced by a bare 300-lb charge of TNT is generated in the reactor compartment from an explosion of 1,000 lb of TNT at the core."

Other arguments based on energy lost by irreversible heating have been advanced and support the contention that such large attenuation is possible.

Thus, in evaluating the effects of a 1,000-lb TNT equivalent nuclear excursion, the air blast resulting from a 300-lb TNT explosion may be used. (This is not to be confused with the 300-lb TNT equivalent nuclear excursion calculated as the lesser accident in the Fermi reactor. A similar reduction in energy would apply to it.) It must be pointed out that the reduction in air blast is not an unencumbered gift. Some of the energy not available for producing air blast would propel fragments of the ruptured material against the surrounding protective structure. Such missiles, however, would be harmful only if they struck the outer containment vessel with penetrating force, which is possible only if they first breach the secondary shield wall or operating floor. The secondary shield wall, then, must be investigated for the effects of the attenuated air blast and internal missiles. The containment vessel must be investigated for the effects of an air blast further attenuated by the secondary shield wall and of any missiles which can be shown to reach the shell. Missiles include those from the reactor region that penetrate the secondary shield wall and those created outside the wall by partial fragmentation of the wall itself or of material between the wall and containment vessel. In this respect R. O. Brittan has said (1),

> "There is little possibility of a shock wave being transmitted to the air between the primary structure (secondary shield wall-authors) and the outer container at the shield interface. A shock loading to the outer shell is not considered in design. At best, only a sudden pressure rise to a calculable maximum can be expected, with a pressure-time relation depending on the exit time of the core materials and flashing or burning times."

As a conservative measure, however, the Fermi containment vessel has been studied for the effect of a 300-lb TNT air blast, representing the initial 70% attenuation of the 1,000-lb TNT equivalent excursion and ignoring the presence of the secondary shield wall. The static pressure rise would not be critical. If we assume a constant-volume adiabatic process with all the energy of detonation of 1,000 lb of TNT (4.54×10^8 calories) effective, the pressure rise would be of the order of 13 psi, much less than the 32 psig for which the vessel was designed.

Reasoning from the foregoing discussion, the following initial guiding principles were adopted for the analysis:

(1) That an excursion of this reactor may be suitably simulated by a high explosive reaction of equivalent energy of detonation.

(2) That cube root scaling of explosion data is valid.

(3) That the positive pressure phase of the blast has a triangular pressure-time relationship.

(4) That rarefaction and secondary shocks may be ignored.

(5) That detonation energy is attenuated by at least 70%. Consequently, the secondary shield wall is considered to experience an air blast equivalent to that of a bare 300-lb TNT charge situated at the reactor center.

(6) That the containment vessel is subjected to the same 300-lb TNT blast, and that the static pressure rise is not critical.

(7) That once containment is assured, under items (5) and (6), consideration must be given to the effects of any internal missiles that may be generated within the reactor building.

SECONDARY SHIELD WALL

Blast Containment.—The monolithic structure composed of the secondary shield wall, the operating floor, and the bottom floor is a highly complex unit whose structural action must be greatly simplified for analysis, and in such a way as to ensure a conservative estimate of damage. The walls and the floors were treated separately.

Dynamic analysis was necessary in view of the large inelastic deformations resulting from high blast pressures. Static analysis, using "equivalent static loads" to replace dynamic loads, is inadmissable since such loads applied for an indefinite time may incorrently indicate arbitrarily large deformations. On the other hand, large loads applied for brief intervals and resisted by inertial accelerations of the structure may result in deformations which, though large, will not rupture the structure, perhaps even leaving it in serviceable condition.

A shock wave, radially propagated from an explosion at the center of the reactor and impinging on the wall, may be visualized as being resisted by interaction of two major types of elements:

1. Vertical strips of the sidewall acting as beams (or one-way slabs) supported at top and bottom.
2. Polygonal hoops or frames as cut by horizontal planes through the sidewalls.

Since these elements act together to carry the blast loading, the capacity of either type alone will be less than the combined capacity; thus, if either type of element can carry by itself the stipulated blast pressure, it may be concluded that the combined action will support a higher blast pressure.

Primary consideration was given to the vertical elements of the wall. They are stiffer than the polygonal hoops and consequently will carry the major portion of the loading and more nearly govern the overall failure mode of the wall. The sidewall was analyzed on the basis of a typical unit strip of a one-way slab, simply supported at top and bottom over a span of nominally 31 ft. The loading on this unit vertical strip was assumed as a uniformly distributed pressure based on the shortest distance from the core to the side-wall.[4]

The vertical strips were analyzed in accordance with the general theory of rigid-plastic beams, which is based on the development of plastic zones at points of maximum stress (11,12,13). Although this theory is well established and thoroughly covered in the literature, a brief review of the basic theory will be included.

Consider a statically applied uniform load on a simply-supported beam, as in Fig. 5. Assume that it is made of mild steel with an idealized stress-strain diagram (tension or compression) as in Fig. 6. The maximum bending moment in the beam occurs at midspan where, under increasing load, the midspan section finally becomes completely plastified. The value of M_{max} when full plastification occurs is known as the "plastic moment capacity," M_p, at which point the beam sags under load and exhibits an abrupt change in curvature at midspan as a result of the large local plastic deformation. Elsewhere the curvature is relatively small, because it is elastic. In essence, then, the beam acts

---

[4] Consideration of non-uniform pressure distribution along the vertical beam is not warranted in view of the other simplifying assumptions of the analysis. In fact, the actual distribution cannot be determined theoretically with any certainty. Usually, the ends of a vertical strip, being at greater radius from the explosion and not at normal incidence with the shock wave, would experience a smaller pressure than the center of the strip. How much smaller is not known, since "the complete analytical solution of even such a relatively simple problem as the behavior of a shock wave incident on a wall at an oblique angle has never been obtained for all angles." On the other hand, shock concentration or focusing in the corners formed by the sidewall with top and bottom slabs increases the pressure in that vicinity and raises it perhaps to the magnitude of the midspan pressure. For a shock running into a 90° corner, "there is a local increase in pressure in the corner of at least four times the initial overpressure in the shock." (Quotations from "The Effects of Atomic Weapons," A.E.C., 1950.) Therefore, an assumption of uniform pressure distribution seems reasonable for the purposes of this study.

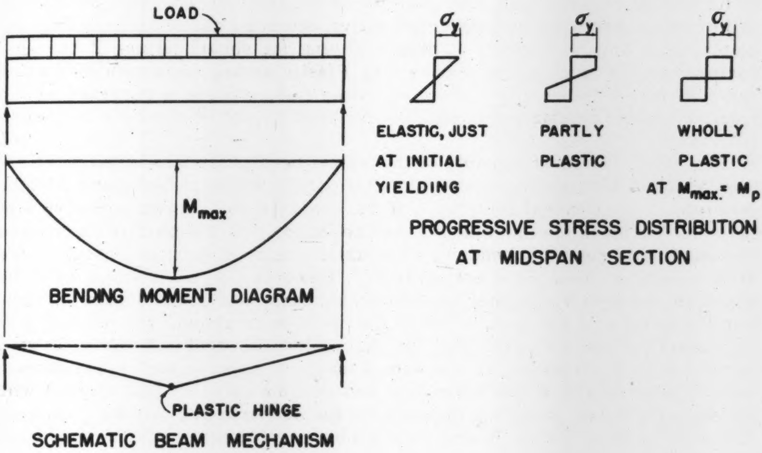

LOAD
M_{max}
BENDING MOMENT DIAGRAM
PLASTIC HINGE
SCHEMATIC BEAM MECHANISM
σ_y
σ_y
σ_y
ELASTIC, JUST AT INITIAL YIELDING
PARTLY PLASTIC
WHOLLY PLASTIC AT $M_{max} = M_p$
PROGRESSIVE STRESS DISTRIBUTION AT MIDSPAN SECTION

FIG. 5.—PLASTIFICATION OF A SIMPLE-SUPPORTED STEEL BEAM

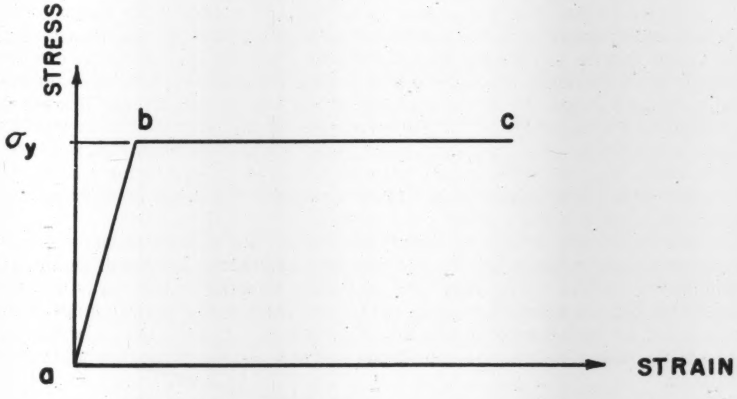


FIG. 6.—IDEALIZED STRESS-STRAIN DIAGRAM FOR STEEL

as two rigid halves connected at midspan by a hinge that allows free rotation once M_{max} has reached M_p. At that point, the beam deflects without limit with no further increase of load, and becomes an unstable mechanism. In addition to the case of plastic bending described above, two other cases were considered, employing a similar concept of plastic sliding (or shear) failure at the supports where maximum shear occurs (12). Plastic sliding was considered alone and combined with bending. The controlling failure mode is governed by the relative plastic resistance in sliding and bending and by the ratio of load to resistance.

The plastic-moment capacity of the typical beam strip was based on an effective cross section consisting of only the steel surface plates unaided by the concrete. The flexural resistance of the concrete was discounted in view of the axial tension in the sidewall and the likelihood of the concrete being cracked and inactive in resisting bending. The axial tension occurs as a result of the blast loading on the top and bottom floors. However, the basic integrity of the concrete, assured by its internal reinforcing bars, was counted upon to maintain the spacing of the surface plates and their beam action. The plastic sliding capacity of the beam was based on the reinforcing steel only, acting in pure tension at its yield point. This assumes that it is possible for the reinforcing bars to bend locally at the beam ends and become more or less aligned with the shearing force, resisting the shear at the supports by tension in the bars. The steel surface plates, having insufficient embedment or attachment, were not counted upon for end resistance. In effect, then, the wall was considered as though it were a beam in which the flanges (steel plates) resist all of the bending and the web (reinforcing bars, concrete, etc.) carries the end reaction only.

Under a dynamic loading of large magnitude the beam first deforms elastically and then shortly thereafter develops plastic "hinges" or "slides" leading to a collapse mechanism. A constant load would cause arbitrarily large deformations and collapse once M_p had been exceeded. However, the blast loading decreases rapidly to zero and is continuously resisted by inertial forces developed in accelerating the beam and by plastic resistances, so that beam motion eventually stops. It was the purpose of the analysis to determine whether or not motion stops short of complete collapse of the beam and to predict the order of magnitude of the permanent deformation when the beam comes to rest. Some simplification of the theory may be achieved by neglecting the elastic deformation and assuming that the plastic zones of the beam exist before and during the complete blast pulse.

It was found that failure by simple plastic sliding at the supports is inadmissible for the relative sliding and bending resistances provided. On the assumption of sliding motion only, the maximum bending moment was found to exceed the plastic moment capacity, which indicates that a plastic hinge would form and that the assumed failure mode is not valid. It was also found that the assumption of a plastic hinge at midspan and non-sliding supports could not be realized.

The governing mode of behavior may be shown to be a combination of plastic hinge at midspan and plastic sliding at the supports. A summary of the calculations for this mode follows.

A beam strip such as that investigated has an infinite number of degrees of freedom. However, by assuming complete rigidity between plastic zones and replacing the actual mass of the beam by an equivalent mass, the equations of

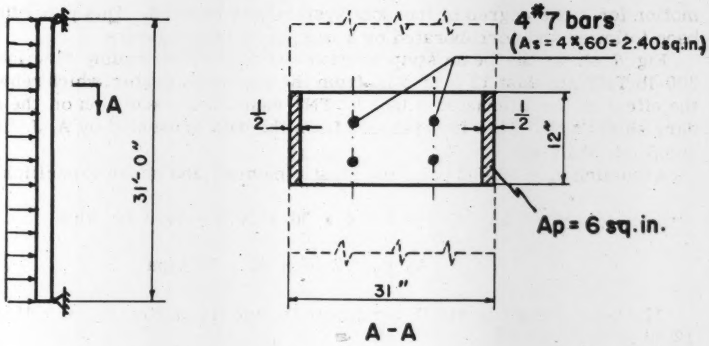

A
A
P
31'-0"
4#7 bars
(As = 4 × .60 = 2.40 sq.in.)
1/2
1/2
12"
Ap = 6 sq.in.
31"
A-A

FIG. 7.—BEAM STRIP

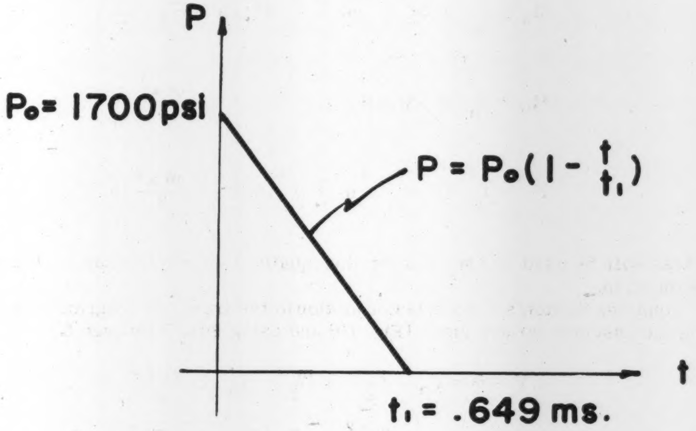


FIG. 8.—LOADING FUNCTION, 300 POUND TNT BLAST

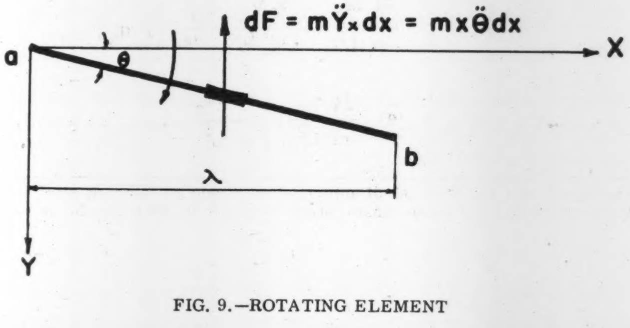

dF = mŸx dx = mxθ̈dx
X
a
θ
b
λ
Y

FIG. 9.—ROTATING ELEMENT

motion for a one degree of freedom system may be used. This procedure has been followed and corroborated by a number of investigators.

Fig. 7 shows the beam strip section and Fig. 8 the loading function for a 300-lb TNT air blast 12 ft by 9 in. from the explosion center, which represents the effect of the attenuated 1,000-lb TNT equivalent excursion on the secondary shield wall. This is developed from the data presented by A. J. Hoffman and S. M. Mills (6).

Assuming $\sigma_y = 30{,}000$ psi,[5] the plastic moment and shear capacities are

$$M_p = A_p \ \sigma_y \ 30.5 = 6 \times 30 \times 30.5 = 5490 \text{ in.-kips} \quad \ldots\ldots \quad (1)$$

$$V_p = A_s \ \sigma_y = 2.40 \times 30 = 72 \text{ kips} \quad \ldots\ldots\ldots\ldots \quad (2)$$

The beam weight is 415 lb per linear ft, and its unit mass, $m = 415/32.2 = 12.89$ lb sec^2 per ft^2.

The general forces due to rotary inertia of a rotating strip of length λ (Fig. 9) are

$$M_a = \int_o^\lambda x \, dF = m \ddot{\theta} \int_o^\lambda x^2 \, dx = \frac{m \lambda^3}{3} \ddot{\theta} \quad \ldots\ldots\ldots \quad (3)$$

$$M_b = \int_o^\lambda (\lambda - x) \, dF = \frac{m \lambda^3}{6} \ddot{\theta} \quad \ldots\ldots\ldots \quad (4)$$

$$F = \int_o^\lambda dF = m \ddot{\theta} \int_o^\lambda x \, dx = \frac{m \lambda^2}{2} \ddot{\theta} \quad \ldots\ldots\ldots \quad (5)$$

These will be used in establishing the equations of equilibrium of the actual beam strip.

Applying Newton's second law of motion to the free body diagram representing the assumed failure mode (Fig. 10) and using Eqs. 3 through 5

$$\Sigma V = 0 = V_p - \frac{p L}{2} + \frac{m L}{2} \ddot{y}_a + \frac{m L^2}{8} \ddot{\theta} \quad \ldots\ldots\ldots \quad (6)$$

$$\Sigma M_a = 0 = M_p - \frac{p L^2}{8} + \frac{m L^2}{8} \ddot{y}_a + \frac{m L^3}{24} \ddot{\theta} \quad \ldots\ldots \quad (7)$$

Solving for $\ddot{y}_a$ and $\ddot{\theta}$ leaves

$$\ddot{y}_a = \frac{p}{m} + \frac{24 M_p}{m L^2} - \frac{8 V_p}{m L} = \frac{1}{m} \left(p - q_a\right) \quad \ldots\ldots\ldots \quad (8)$$

$$\ddot{\theta} = \frac{24 V_p}{m L^2} - \frac{96 M_p}{m L^3} \quad \ldots\ldots\ldots\ldots\ldots \quad (9)$$

---

[5] The specified yield strength of intermediate grade reinforcing steel is 40,000 psi, but 30,000 psi was used as a conservative measure in studying the secondary shield wall.

In addition,

$$\ddot{y}_c = \ddot{y}_a + \frac{L}{2}\ddot{\theta} = \frac{p}{m} - \frac{24\ M_p}{m\ L^2} + \frac{4\ V_p}{m\ L} = \frac{1}{m}\left(p - q_c\right) \quad \ldots\ldots (10)$$

in which

$$q_a = \left(-\frac{24\ M_p}{L^2} + \frac{8\ V_p}{L}\right) \quad \ldots\ldots\ldots\ldots (11)$$

and

$$q_c = \left(\frac{24\ M_p}{L^2} - \frac{4\ V_p}{L}\right) \quad \ldots\ldots\ldots\ldots (12)$$

Eqs. 8 and 10 are ordinary differential equations in which p represents the driving force (loading function) as defined in Fig. 8, and q represents the structural resistance as defined in Eqs. 11 and 12.

In general terms, the solution of the equation

$$\ddot{y} = \frac{1}{m}\ (p - q) \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

is, for $0 \leq t \leq t_1$

$$\dot{y} = \frac{t}{m}\left[p_o\left(1 - \frac{t}{2\ t_1}\right) - q\right] \quad \ldots\ldots\ldots\ldots (14)$$

$$y = \frac{t^2}{2\ m}\left[p_o\left(1 - \frac{t}{3\ t_1}\right) - q\right] \quad \ldots\ldots\ldots\ldots (15)$$

and, for $t \geq t_1$

$$\ddot{y} = -\frac{q}{m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

$$\dot{y} = \frac{1}{m}\left[\frac{p_o\ t_1}{2} - q\ t\right] \quad \ldots\ldots\ldots\ldots\ldots (17)$$

$$y = \frac{1}{2\ m}\left[p_o\ t_1\left(t - \frac{t_1}{3}\right) - q\ t^2\right] \quad \ldots\ldots\ldots\ldots (18)$$

If the loading and resistance are such that outward motion stops prior to the time t_1, then t_{max}, the time at which this occurs, can be determined by setting the velocity Eq. 14 equal to zero.

$$t_{max} = 2\ t_1\left[1 - \frac{q}{p_o}\right] \quad \ldots\ldots\ldots\ldots (19)$$

which is valid only for $t_{max} \le t_1$, or when

$$\frac{q}{p_o} \ge \frac{1}{2} \tag{20}$$

The total deflection corresponding to t_{max} is

$$y_{max} = \frac{t_{max}^2}{6\ m} \left(p_o - q\right) \tag{21}$$

If, on the other hand, $q/p_o \le 1/2$, then outward motion stops at a time greater than t_1, namely

$$t_{max} = \frac{p_o\ t_1}{2\ q} \tag{22}$$

and the total deflection is

$$y_{max} = \frac{p_o\ t_1^2}{2\ m} \left[\frac{p_o}{4\ q} - \frac{1}{3}\right] \tag{23}$$

By substitution of the following values: M_p = 5490 in.-kips, V_p = 72 kips, L = 31 ft and p_o = 1700 psi, into Eqs. 11, 12 and 20, it is determined that q_a = 599 lb per in. and q_c = 177 lb per in. and also that $\frac{q_a}{p_o} + \frac{q_c}{p_o} \ll \frac{1}{2}$. Therefore, $t_{max} > t_1$. From Eq. 23

$$y_{c\ max} = \frac{1700 \times 144 \times 0.649^2 \times 32.2}{2 \times 415 \times 10^6} \left(\frac{20,400}{4 \times 177} - \frac{1}{3}\right) = 0.110 \text{ ft} = 1.31 \text{ in.}$$

and

$$y_{a\ max} = 4.0 \times 10^{-3} \left(\frac{20,400}{4 \times 599} - \frac{1}{3}\right) = 0.033 \text{ ft} = 0.39 \text{ in.}$$

also

$$y_{net} = y_c - y_a = 0.92 \text{ in.}$$

In order for the above solution to be valid the shear, V_x, and the bending moment, M_x, must be less than the limiting values V_p and M_p at all points in the beam except at the respective plastic zones assumed in the analysis. These conditions may be checked by using the equations for shear and bending moment. From Fig. 10, and using Eqs. 3, 4 and 5

$$V_x = V_p - p\ x + m\ x\ \ddot{y}_a + \frac{m\ x^2}{2}\ \ddot{\theta} \tag{24}$$

Substituting for $\ddot{y}_a$ and $\ddot{\theta}$ from Eqs. 8 and 9

$$V_x = \left(1 - \frac{2\ x}{L}\right) \left[V_p \left(1 - \frac{6\ x}{L}\right) + 24\ \frac{M_p\ x}{L^2}\right] \tag{25}$$

Similarly,

$$M_x = V_p\ L\left(\frac{x}{L}\right)\left(1-\frac{2\ x}{L}\right)^2 + 4\ M_p\left(\frac{x}{L}\right)^2\left(3-\frac{4\ x}{L}\right) \quad \ldots (26)$$

These are valid for any time, t.

From Eq. 25, points of zero shear and maximum bending moment occur at

$$x_1 = L/2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (27a)$$

$$x_2 = \frac{V_p\ L}{6\left(V_p - 4\frac{M_p}{L}\right)} \quad \ldots\ldots\ldots\ldots (27b)$$

The solution is valid only if $x_2 \geq \frac{L}{2}$; otherwise plastic hinges will form first at points $x_2 < L/2$, thus violating the assumed mode of behavior. The criterion for $x_2 \geq L/2$ may be written as

$$\frac{V_p\ L}{2\ M_p} \leq 3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

For the case being studied, $\frac{V_p\ L}{2\ M_p} = 2.44 < 3$; therefore, the bending moment criterion is satisfied. A similar shear criterion is also satisfied and the assumed mode of behavior is valid.

The maximum deflection is less than 1/280th of the span and the maximum net deflection is less than 1/400th of the span. It may be concluded that the wall will withstand blasts from as much as 1,000 lb of TNT attenuated 70% and sustain only minor damage. Since the vertical elements will be assisted in carrying blast loads by the horizontal polygonal hoop elements, the actual blast damage would be insignificant and restricted largely to cracking of the periphery of the flat reinforced concrete plate elements forming the wall. Neglecting any blast attenuation, that is, assuming the full effect on the face of the wall of a 1,000-lb TNT bare charge, it was found that the net deflection would be less than 1/100th of the span. Even under this impossible condition the wall would not collapse.

The end reaction of the wall under the conditions of motion described above and taken as a constant force of V_p, or 72 kips per ft, would act outward on the floors above and below the wall for the duration of motion of the ends of the walls. From Eq. 22, this time would be, for the attenuated blast

$$t_1' = \frac{p_o\ t_1}{2\ q_a} = \frac{1700 \times 12 \times 0.649}{2 \times 599} = 11.06 \text{ msec}$$

At the end of this time, the force may be considered to drop abruptly to zero giving the loading function shown in Fig. 11.

It may be visualized that this force acts in the plane of the floor slabs that provide the support against outward radial movement of the side-walls. The floors, subjected to this more or less radially-applied force, tend to tear apart along vertical radial planes with consequent danger that the sectors of the broken floor may move outward against the containment vessel. The manner in which this bursting tendency is resisted, particularly in the operating floor, is extremely difficult to predict. For simplified and conservative analysis the operating floor may be considered as a hoop which resists bursting through tension in its several components: concrete, reinforcing, steel plates on the underside, and structural supporting members.

To obtain an idea of the amount of floor reinforcement necessary to prevent excessive cracking, a dynamic analysis was made assuming an equivalent hoop of reinforcing steel with a radius of 20 ft. It was assumed that this hoop may be expected to carry only half of the total reaction because of the effective restraint of the other components previously mentioned, the fact that the secondary shield wall itself acts somewhat as a polygonal hoop, and the fact that the load intensity assumed is that for the point closest to the reactor center line, whereas most points are at greater distances. Crossing all radial sections at approximately 20 ft from the building center is the equivalent of at least 20 sq in. of intermediate grade reinforcing steel. This value was used in the analysis.

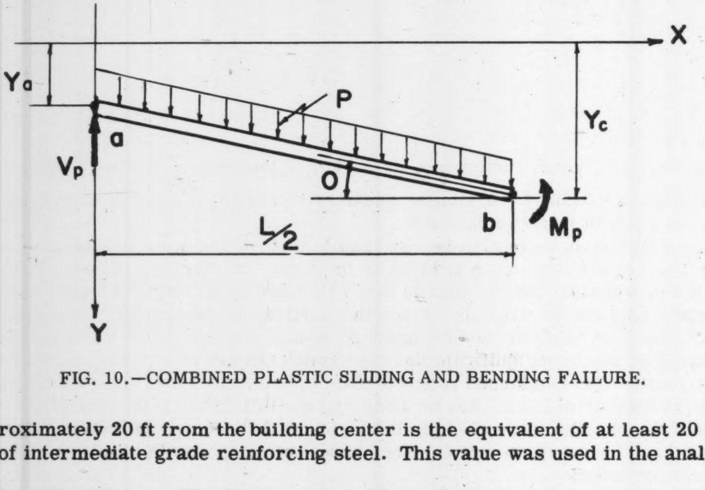


FIG. 10.—COMBINED PLASTIC SLIDING AND BENDING FAILURE.

Assuming the simplified hoop action of Fig. 12, it may be shown that, while the steel remains elastic the radial displacement is

$$x = \frac{V\ r^2}{E\ A} \quad (1 - \cos\ \omega\ t) \quad \ldots\ldots\ldots\ldots\ (29)$$

in which A is the area of steel in hoop,

$$\omega = \sqrt{\frac{E\ g}{\gamma\ r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (30)$$

and γ is the specific weight of steel.

When, at time $t_e \leq t'_1$, the steel becomes plastic, the displacement equation is

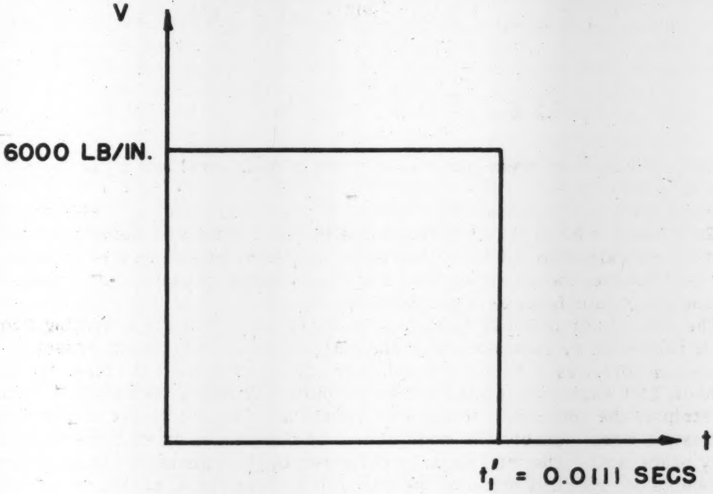


FIG. 11.—FORCE-TIME DIAGRAM FOR OPERATING FLOOR

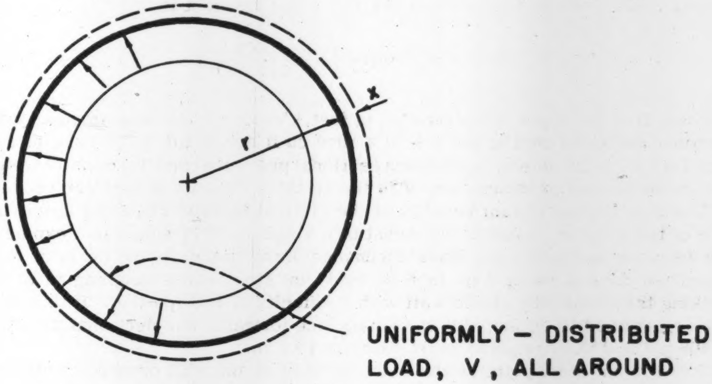

x
r
UNIFORMLY — DISTRIBUTED
LOAD, V, ALL AROUND

FIG. 12.—HOOP ACTION OF OPERATING FLOOR REINFORCEMENT

$$x = \frac{V\ r^2}{E\ A} \left[\left(t - t_e\right) \omega \sin \omega t_e + 1 - \cos \omega t_e \right]$$

$$+ \frac{1}{2\ m} \left(V - \frac{\sigma_y\ A}{r}\right)\left(t - t_e\right)^2 \qquad \text{. . . . . (31)}$$

in which m refers to mass per running inch of hoop steel and σ_y is the yield point of steel.

Using the conditions that $V = 3$ k per in., $t'_1 = 0.111$ sec, $\sigma_y = 40{,}000$ psi, $r = 20$ ft and $A = 20$ sq in., it is found that the total computed radial displacement is approximately 1.6 in. This is not excessive since there is an expansion joint between the operating floor and the containment vessel, and a moderate amount of interference is permissible.

The lower floor is not critical. It is more massive than the operating floor and is backed up by concrete fill on the outside of the containment vessel.

Internal Missiles.—As noted previously, the unattenuated air blast from a 1,000-lb TNT explosion, when considered uniformly distributed along a vertical strip of the secondary shield wall, would not cause collapse of the wall. This is the worst conceivable condition to be considered in studying the stability of the wall. The total impulse delivered by the attenuated air blast plus fragments or missiles striking the wall at the same time, cannot be greater than that of the parent blast on the same area.

Several theories on the local piercing action of missiles have been developed but they all require, for quantitative application, the experimental determination of the penetration-resisting properties of the material struck. The best data of this type are those obtained by the armed services in their tests of bomb-resistant structures.

The penetrating effect of a missile can be estimated by the modified Petry formula, used by the Bureau of Yards and Docks, Dept. of the Navy, for designing bomb resistant structures (14,15). This formula is

$$D = k\ A_p \log_{10} \left(1 + \frac{V^2}{215{,}000}\right) \qquad \text{. . . . . (32)}$$

in which D is the depth of penetration in feet, k refers to an experimentally determined material coefficient = 4.76×10^{-3} cu ft per lb for 3,000 psi concrete with 1.4% reinforcement, A_p denotes sectional pressure, that is, missile weight divided by frontal pressure, and V is the striking velocity in feet per second.

The size and maximum velocity of the critical missile resulting from rupture of the reactor vessel in the event of a 1,000-lb TNT equivalent explosion has been estimated by the Naval Ordnance Laboratory on the basis of their classified data as being 2 in. by 6 in. by 10 in. steel block weighing 34 lb and striking the secondary shield wall with a velocity of 903 fps (10). If this missile is assumed to strike with the small face forward, the sectional pressure is 408 psf and the computed penetration is 15.8 in.

The 1/2-in. steel plate on the inner surface of the wall provides additional penetration resistance. A reasonable estimate is that structural steel is twelve times as effective as the same thickness of concrete (15), thereby reducing the expected penetration in concrete to about 10 in. Since this is not greater than one-third the total wall thickness, the wall cannot be breached.[6] Furthermore, the beneficial effect of the primary shield graphite is neglected.

There is additional assurance of the wall's ability to withstand perforation in the results of a bomb shelter test conducted by the Navy Dept. in 1940 (14). A 6-in., 100-lb, pointed projectile was fired at a 31-1/2-in. thick reinforced concrete roof slab at an angle of 21°30', with a striking velocity of 1,023 fps. The slab was of 4,500 psi concrete without steel face plates and had three layers of reinforcing bars in each direction: 5/8-in. round bars at 5 in. spacing one way, 5/8-in. round bars at 10 in. the other. The sectional pressure of the missile was 513 psf. While not identical to the problem under investigation here, in most respects the conditions were remarkably similar and results of the same order of magnitude can be expected. To quote from the Navy report:

> "The 6-inch projectile penetrated the 31 1/2-in. outer roof slab 14 1/2 in., rebounded, and fell in front of the shelter. There were no visible cracks or scabbing on the ceiling inside the shelter. The deflection gage showed a maximum reading of 0.01 in."

The span of the slab was about 16 ft.

In addition to the general correlation between observed and computed depths of penetration, the absence of scabbing on the opposite side of the slab is important since it bears on the question of whether missiles might be formed outside of the secondary shield wall by scabbing of the concrete on the outer side of the secondary wall. The Navy test report indicates that this will not happen even for an unprotected wall. For a wall covered on the back by firmly anchored 1/2-in. steel plates there is no question of scabbing or of rupture of the outer plate. The Navy report states further that anti-scabbing plates were effective in increasing perforation resistance in their tests. It may be concluded that breaching or scabbing of the secondary shield wall or the thicker operating floor above is not a problem. Furthermore, the few random missiles which might approach the 10-in. penetration depth at intervals on the wall would not impair the overall dynamic strength of the wall because of their localized effect.

CONTAINMENT VESSEL

Design.—Design and construction of the containment vessel follow the best current practices:

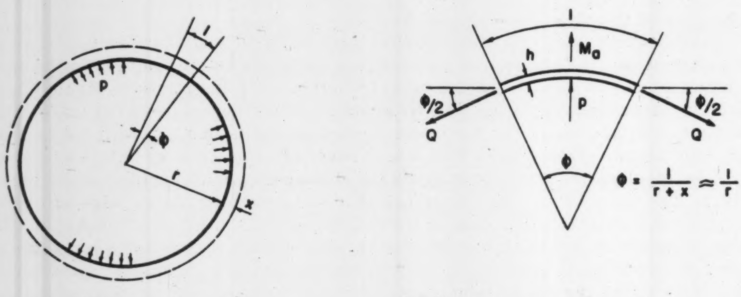
1. The hemiellipsoidal bottom is completely encased in a massive concrete mat bearing directly on rock. There is no possibility of settlement which would cause unforeseen stresses in the vessel.
2. There is a transition section between the full encasement and the free section of the vessel which will prevent the development of large bending stresses possible at a sharp discontinuity in supports.
3. The steel fabricators who designed the vessel have accounted for all reasonable operating loads (dead, crane, wind, snow and temperature). They have also checked the capacity of the structure to resist an earthquake of very heavy magnitude for the Michigan area. They have accounted for bending at constraints as well as primary membrane stresses.

---

[6] Bibliography reference (15) states that the Petry formula is directly applicable only when the computed penetration is less than 1/3 the thickness of the wall. For computed penetrations greater than 1/2 the wall thickness, breaching may occur.

4. The designers have accounted for the maximum internal pressure of 32 psig originally specified as that which might be expected in the event of a sodium fire. They have also provided for a specified external pressure of 2 psig with vacuum relief for pressures above this, another condition which might result from a sodium fire. It would appear that the use of allowable stresses specified by the ASME code for vessels under pressure during normal operation is a conservative practice for containment vessels which are under pressure only in the event of an unlikely accident.

5. The use of normalized killed steel with a minimum Charpy impact resistance of 15 ft-lb at -50° F, full penetration butt welding and stress relieving wherever possible, and the avoidance of serious stress raisers, afford the best means of protection known at present against brittle fracture, and provide excellent resistance against normal operating loads or emergency condition dynamic loads. Several exhaustive surveys of brittle failure in actual structures have brought to light no such failures in the type of steel used for the containment vessel (16,17).



(a) SECTION OF CYLINDER

(b) UNIT ELEMENT

FIG. 13.—ACTION OF CONTAINMENT VESSEL

Blast Containment.—Little or no air blast effect is expected on the outer shell, but the shell was analyzed for the blast loading from a 1,000-lb TNT explosion, attenuated 70%, to obtain an extreme upper limit of damage.

It was assumed that any blast wave could exert maximum pressure on the shell in the area below the operating floor. In this region the shell is a simple cylinder. It may be treated as an elemental hoop dynamically loaded by the internal shock wave. The vertical stresses in the shell wall have been neglected in the dynamic containment analysis. Consider a unit length of shell (Fig. 13). Summing forces in the radial direction

$$p - 2\,Q\,\frac{\phi}{2} = M\,a \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (33)$$

$$p - \frac{Q}{r} = M\,a \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (34)$$

Assuming a triangular pressure pulse,

$$p = p_o \left(1 - t/t_1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

and

$$\begin{aligned} Q &= \sigma h \\ M &= h(\gamma/g) \\ a &= d^2x/dt^2 \end{aligned}$$

in which σ is the unit stress in the ring, σ refers to the $E\epsilon$ for elastic range, σ is the σ_y (yield point) for plastic range, and γ = specific weight of steel. Then, by substitution in the force equation,

$$\frac{d^2x}{dt^2} + \frac{g\,\sigma}{\gamma\,r} = \frac{p_o\,g}{\gamma\,h}\left(1 - \frac{t}{t_1}\right) \quad \ldots\ldots\ldots\ldots\ldots (36)$$

Integration of this equation and application of the boundary conditions yields two sets of equations for the radial displacements, x, as a function of time: one if t_1 is less than the time, t_e, at which elastic action stops and yielding begins, and the other if t_1 is greater than t_e. For the first case, which was found to hold, the equations are:

Elastic action.—$0 \leq t \leq t_1$

$$x = \frac{p_o\,r^2}{E\,h}\left[\frac{1}{\omega\,t_1}\sin\omega t - \cos\omega t + \left(1 - \frac{t}{t_1}\right)\right] \quad \ldots . (37)$$

Elastic action.—$t_1 \leq t \leq t_e$

$$x = \frac{p_o\,r^2}{E\,h}\left[\frac{1}{\omega\,t_1}\left(1 - \cos\omega t_1\right)\sin\omega t - \left(\frac{\omega t_1 - \sin\omega t_1}{\omega t_1}\right)\cos\omega t\right] \quad .. (38)$$

Plastic action.—$t_e \leq t$

$$\begin{aligned} x = \frac{p_o\,r^2}{E\,h} &\left\{\left[\frac{(t - t_e)}{t_1}\cos\omega t_e + \frac{\sin\omega t_e}{\omega\,t_1}\right]\left[1 - \cos\omega t_1\right]\right. \\ &\left. + \left[\frac{(t - t_e)}{t_1}\sin\omega t_e - \frac{\cos\omega t_e}{\omega\,t_1}\right]\left[\omega t_1 - \sin\omega t_1\right]\right\} \\ &- \frac{\sigma_y\,g}{2\,\gamma\,r}\left(t - t_e\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (39) \end{aligned}$$

in which

$$\omega = \sqrt{\frac{E\,g}{\gamma\,r^2}}$$

For a 300-lb TNT blast (6), for a radius of 36 ft:

Face on overpressure, p_o = 84 psi
Face on specific impulse = 0.182 lb. sec/in.^{2}
Assuming linear pressure decay, t_1 = 4.35 ms.
σ_y = 32,000 psi
E = 30 x 10^6 psi
h = 1.03 in. = shell thickness

It was found from the preceding equations that the radial expansion would stop while the shell is still elastic at a time of 4.78 msec. The maximum expansion would be 0.459 in., and the computed maximum stress 31,800 psi. Thus the containment vessel would not yield but would vibrate radially within the stated range until damped out. For this condition the maximum rate of strain may be shown to be 0.34 in. per in. sec. This is not an excessive strain rate of the type that might produce fracture. At this rate an increase in the yield point of at least 15% might be expected (18), further guaranteeing elastic behavior.

ADDITIONAL EFFECTS

In addition to the blast effects summarized previously, the following problems were studied:

1. The motion of the rotating plug, control rods, and materials handling equipment caused by air blast or static pressure rise within the secondary shield wall enclosure following an explosion. It was found that none of these objects could be converted into missiles.
2. The transmission of shock waves through the sodium piping and the tendency to fracture it and convert it into missiles. This was found not to be a problem.
3. The effect of air blasts on equipment outside the secondary shield enclosure, that is, heat exchangers, sodium pumps, piping, the overhead crane, etc. It was found that these would not be detached from their supports.
4. The effect of a sodium fire in heating exposed steel surfaces to the danger point. Recommendations were made for moderate fireproofing of the steel in certain areas.
5. The possibility of an external missile such as an aircraft or portion of one striking the containment vessel. Probability studies showed that this is a negligible factor. Even if one did hit, it would have to be extraordinarily heavy or have a very high velocity to penetrate below the operating floor where it might damage the reactor.

CONCLUSIONS

The major conclusions regarding blast effects can be summarized. In the event of a nuclear excursion equivalent to the detonation of 1,000 lb of TNT at the reactor core the following is expected to happen:

1. The blast wave will deform the secondary shield wall permanently. It may bend and yield at about mid-height and will crack along the edges. The steel face plates and the embedded structural and reinforcing steel will keep the permanent deformations within tolerable limits.

2. The operating floor may crack because of the dynamic loads from the secondary shield wall. However, with the reinforcement provided, the outward movement of the operating floor will not endanger the containment vessel.

3. The reactor, reactor vessel, sodium piping, borated graphite, and primary shield tank in the region of the reactor core may fracture and form missiles. However, these missiles will be contained within the secondary shield wall and will not endanger the containment vessel.

4. The shock wave will not cause the rotating plug and control rods to form dangerous missiles.

5. Missiles will not be formed outside of or above the secondary shield wall region.

6. There will be very little if any shock effect on the outer containment vessel. It is capable of containing any possible residual shock without deforming plastically.

7. The post-blast static pressures and temperatures in the outer vessel cannot be as serious as those for which the vessel was designed.

ACKNOWLEDGMENTS

The study summarized in this paper was made by the authors as consultants to Power Reactor Development Company, builders and operators of the Enrico Fermi Plant. Design of the reactor and initial testing prior to nuclear operation are under the jurisdiction of Atomic Power Development Associates, Inc., the associated design and development organization. Output will be delivered to an adjacent 150 MW turbine - generator plant being built by the Detroit Edison Company. The structural design was prepared by Commonwealth Associates, Inc., with the exception of the outer steel containment vessel, designed and erected by the Chicago Bridge and Iron Company. Additional studies were made by the Cornell Aeronautical Laboratory.

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Paper No. 3132

REGIONAL PLANNING IN CUYAHOGA COUNTY, OHIO

By Alfred A. Estrada,[1] F. ASCE

SYNOPSIS

The paper points out the problems created in the fields of water supply and sewerage by the post-war population boom. The manner in which the problem was studied and the observations that were made are examined.

THE PROBLEM

The rapid post-war growth, both in population and in industrial activity, of the Cleveland, Ohio, metropolitan area has brought with it many problems. Not the least among these are problems involving water supply, sewage collection and disposal, and storm-water drainage. Facilities which were adequate a few years ago are now inadequate. Sewage treatment plants are, or are becoming, overloaded and must be enlarged or rebuilt. Areas where septic tanks were formerly sufficient must now have sewers and treatment plants. Discharge of raw sewage into the streams and into Lake Erie, tolerated at one time, is now prohibited. Heavier storm water run-off, due to development of land and increasing property values, have resulted in greatly increased damages and inconveniences due to flooding. A program of construction of sanitary and storm water facilities is urgently needed.

Added to the need for these facilities is another problem. Whereas the population was formerly concentrated largely in the City of Cleveland, it is now spreading all over the County, beyond the city limits. The sanitation problem concerns not only the City of Cleveland, but, also, the entire County. There is no single existing governmental agency which is adequate to meet

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Note.—Published essentially as printed here, in May, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2471. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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the problem. Sewerage facilities involving several municipalities must be built. In many places, flooding occurs in one area from storm water falling in another municipality. Some flood-control facilities must be built which will benefit several communities. Construction of such regional projects is retarded because of inadequate financial and administrative facilities. The need for some type of regional district with the necessary authority is, thus, indicated.

STATUS OF PROGRAM

To solve these problems, in 1948, the Regional Planning Commission began a comprehensive study of the metropolitan water and sewerage problems. Since then, a development program has been undertaken and the following steps have been carried out:

1. In 1949, the General Assembly of the State of Ohio adopted enabling legislation in the form of the "Regional Water and Sewer Act."
2. In 1950, a $500,000 County bond issue was voted to finance a master water and sewer plan.
3. The Regional Planning Commission was authorized to undertake preparation of the plan and established a coordinating committee of engineers to supervise the project.
4. The firm of Wainwright and Ramsey of New York, N.Y., was engaged as financial consultant.
5. In 1953, a report, entitled "The Sewer and Water Problem," was prepared by the Regional Planning Commission.
6. The firm of Havens and Emerson, Consulting Engineers, of Cleveland Ohio, was engaged to prepare a master water plan, which was completed in August, 1953.
7. The Real Property Inventory, a statistical organization of Cuyahoga County, was engaged to prepare population forecasts.
8. For preparation of the master sewerage plan, the County was divided into four parts, and each part was assigned to a local engineering office, designated as an "Area Engineer." In connection with this work, the Planning Commission obtained aerial topography of the critical portions of the County. The reports of these Area Engineers were completed in 1956 and 1957.
9. The firm of Albright and Friel Inc., was engaged, in 1956, to coordinate the reports of the four Area Engineers. This work was to insure the orderly development of integrated systems for the collection and disposal of sewage and the handling of storm water within Cuyahoga County. The work was, also, to include the devising of a workable plan for financing, administration, and operation of the integrated systems. The report covering this assignment was completed on December 30, 1956, and this paper will summarize its findings.

SANITARY SEWERS

The first major function of the assignment was to review sanitary projects studied by the Area Engineers.

The Area Engineers studied some 35 sanitary sewerage projects which were estimated to have a construction cost of approximately $104,000,000.

Twenty-eight of the projects were considered by the Area Engineers for immediate construction and these were reduced to 25 projects due to consolidations by the restudy.

It has been conceived that a central agency such as a proposed Regional District would finance the construction of only those portions of the projects which would be considered regional in scope. The costs have been separated into local sewers and the regional parts of the project defined as main intercepting sewers and sewage treatment works. Sewers included in the Area Engineers' study, which have been considered local sewers, include branch intercepting sewers and branch sewers. For the purposes of this report, main intercepting sewers were considered to be sewers which drain at least 20% of the entire drainage area. This limitation was to be waived in cases where an intercepting sewer must pass through one municipality to serve an upstream municipality. In this case, the boundary of the upstream municipality would be the limit of regional construction.

The 25 projects under consideration for immediate construction are estimated to have a total cost of $46,347,000. These project costs would break down into Regional and Local Costs as follows:

a) Regional Construction	
Main Intercepting Sewers	$17,583,000
Sewage Treatment Plants	18,387,000
Total Regional Construction	$35,970,000
b) Local Construction	
Branch Intercepting Sewers	10,377,000
TOTAL	$46,347,000

STORM DRAINAGE

A second major function of the assignment was to review the work of the Area Engineers who took an inventory of existing drainage systems and who designed necessary improvements. The Area Engineers studied numerous improvement projects estimated to cost a total of some $74,000,000. These projects included many items of construction which were considered to be of a local nature. In order to differentiate between local and regional, a limit of construction was adopted for this report establishing that the Regional District would construct only the portion of projects which had a 1/2 sq mile (320 acres) or more of drainage area.

After adjusting the work studied by the Area Engineers for the regional limitation and adding nine projects which were studied individually by Albright and Friel Inc., the total program studied was condensed to 51 projects with a total estimated cost of $42,943,000.

This total program contemplates the construction of 16 storm trunk projects, 21 single improvement projects, 7 stream enclosures, 2 projects involving general improvement and stream enclosures, 4 retarding basins, and the modification of one existing retarding basin.

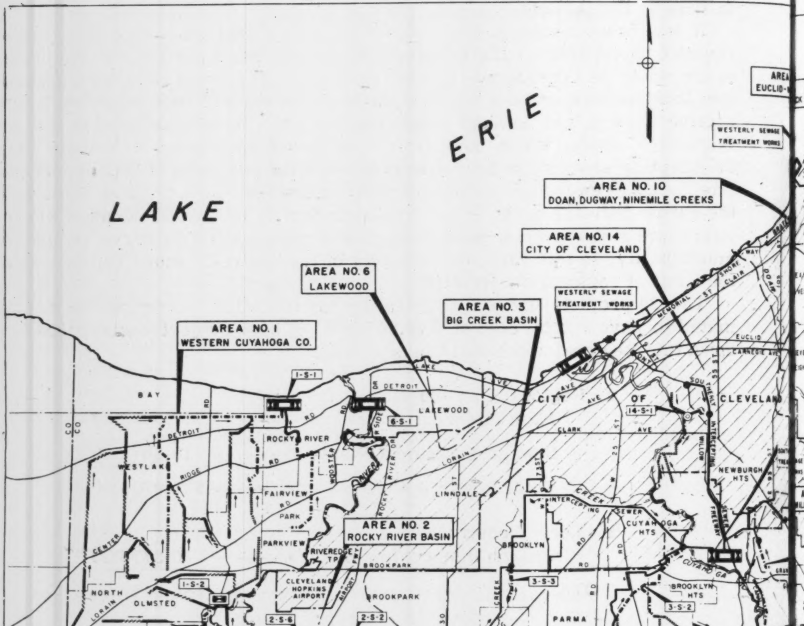
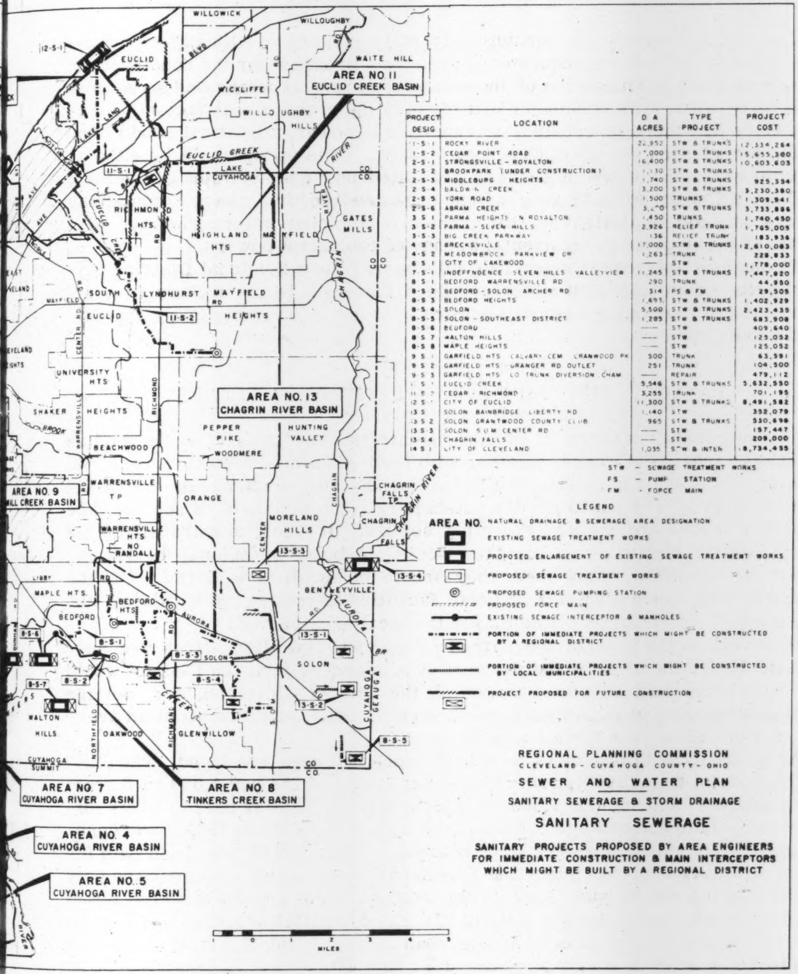

LAKE
ERIE
AREA NO. 10
DOAN, DUGWAY, NINEMILE CREEKS
AREA NO. 14
CITY OF CLEVELAND
AREA NO. 6
LAKEWOOD
AREA NO. 3
BIG CREEK BASIN
AREA NO. 1
WESTERN CUYAHOGA CO.
AREA NO. 2
ROCKY RIVER BASIN
WESTERLY SEWAGE TREATMENT WORKS
1-S-1
1-S-2
6-S-1
14-S-1
2-S-1
2-S-2
2-S-3
2-S-4
2-S-5
2-S-6
3-S-1
3-S-2
3-S-3
4-S-1
4-S-2
BAY
WESTLAKE
ROCKY RIVER
LAKEWOOD
FAIRVIEW PARK
PARKVIEW
RIVEREDGE TP
CLEVELAND HOPKINS AIRPORT
BROOKPARK
LINNDALE
BROOKLYN
CUYAHOGA HTS
BROOKLYN HTS
NEWBURGH HTS
CITY OF CLEVELAND
PARMA
PARMA HTS
SEVEN HILLS
INDEPENDENCE
BROADVIEW HTS
NORTH OLMSTED
OLMSTED TP
OLMSTED FALLS
BEREA
BEREA S.T.W.
MIDDLEBURG HTS
NORTH ROYALTON
STRONGSVILLE
DETROIT
CENTER RIDGE RD
LORAIN RD
CLARK AVE
BIG CREEK INTERCEPTING SEWER
SOUTHERLY INTERCEPTING
EAST BRANCH ROCKY RIVER
CUYAHOGA RIVER
TURNPIKE
LORAIN CO
CUYAHOGA CO
MEDINA
SUMMIT
EUCLID
CARNEGIE AVE
EAST CLEVELAND
CLEVELAND HEIGHTS
AREA NO.
EUCLID

FIG. 1


WILLOWICK
WILLOUGHBY
WAITE HILL
12-S-1
EUCLID
AREA NO. 11
EUCLID CREEK BASIN
WICKLIFFE
WILLOUGHBY HILLS
EUCLID CREEK
LAKE CUYAHOGA
11-S-1
RICHMOND HTS.
HIGHLAND HTS
MAYFIELD
GATES MILLS
CHAGRIN RIVER
SOUTH EUCLID
LYNDHURST
MAYFIELD HEIGHTS
11-S-2
UNIVERSITY HTS
AREA NO. 13
CHAGRIN RIVER BASIN
SHAKER HEIGHTS
PEPPER PIKE
HUNTING VALLEY
BEACHWOOD
WOODMERE
AREA NO. 9
MILL CREEK BASIN
WARRENSVILLE TP
ORANGE
CHAGRIN FALLS TP
MORELAND HILLS
CHAGRIN FALLS
WARRENSVILLE HTS
NO. RANDALL
13-S-3
13-S-4
MAPLE HTS.
BEDFORD HTS
BENTLEYVILLE
BEDFORD
8-S-6
8-S-1
8-S-3
SOLON
13-S-1
8-S-2
8-S-4
13-S-2
8-S-7
WALTON HILLS
OAKWOOD
GLENWILLOW
CUYAHOGA GEAUGA
8-S-5
CUYAHOGA SUMMIT
AREA NO. 7
CUYAHOGA RIVER BASIN
AREA NO. 8
TINKERS CREEK BASIN
AREA NO. 4
CUYAHOGA RIVER BASIN
AREA NO. 5
CUYAHOGA RIVER BASIN
MILES

PROJECT DESIG	LOCATION	D A ACRES	TYPE PROJECT	PROJECT COST
1-S-1	ROCKY RIVER	22,952	STW & TRUNKS	12,334,264
1-S-2	CEDAR POINT ROAD	17,000	STW & TRUNKS	15,655,300
2-S-1	STRONGSVILLE - ROYALTON	16,400	STW & TRUNKS	10,603,603
2-S-2	BROOKPARK (UNDER CONSTRUCTION)	1,130	STW & TRUNKS	——
2-S-3	MIDDLEBURG HEIGHTS	1,740	STW & TRUNKS	925,354
2-S-4	BALDWIN CREEK	3,200	STW & TRUNKS	3,230,380
2-S-5	YORK ROAD	1,500	TRUNKS	1,309,941
2-S-6	ABRAM CREEK	3,200	STW & TRUNKS	3,733,886
3-S-1	PARMA HEIGHTS N ROYALTON	1,450	TRUNKS	1,740,450
3-S-2	PARMA - SEVEN HILLS	2,926	RELIEF TRUNK	1,765,005
3-S-3	BIG CREEK PARKWAY	136	RELIEF TRUNK	183,936
4-S-1	BRECKSVILLE	17,000	STW & TRUNKS	12,610,083
4-S-2	MEADOWBROOK PARKVIEW DR	1,263	TRUNK	228,833
6-S-1	CITY OF LAKEWOOD	——	STW	1,778,000
7-S-1	INDEPENDENCE SEVEN HILLS VALLEYVIEW	11,245	STW & TRUNKS	7,447,820
8-S-1	BEDFORD WARRENSVILLE RD	290	TRUNK	44,950
8-S-2	BEDFORD - SOLON ARCHER RD	314	PS & FM	29,505
8-S-3	BEDFORD HEIGHTS	1,693	STW & TRUNKS	1,402,929
8-S-4	SOLON	5,500	STW & TRUNKS	2,423,435
8-S-5	SOLON - SOUTHEAST DISTRICT	1,285	STW & TRUNKS	683,908
8-S-6	BEDFORD	——	STW	409,640
8-S-7	WALTON HILLS	——	STW	125,052
8-S-8	MAPLE HEIGHTS	——	STW	125,052
9-S-1	GARFIELD HTS CALVARY CEM CRANWOOD PK	500	TRUNK	63,591
9-S-2	GARFIELD HTS GRANGER RD OUTLET	251	TRUNK	104,500
9-S-3	GARFIELD HTS CO TRUNK DIVERSION CHAM	——	REPAIR	479,112
11-S-1	EUCLID CREEK	5,546	STW & TRUNKS	5,632,550
11-S-2	CEDAR - RICHMOND	3,255	TRUNK	701,195
12-S-1	CITY OF EUCLID	11,300	STW & TRUNKS	8,491,582
13-S-1	SOLON BAINBRIDGE LIBERTY RD	1,140	STW	352,079
13-S-2	SOLON GRANTWOOD COUNTY CLUB	965	STW & TRUNKS	530,698
13-S-3	SOLON S O M CENTER RD	——	STW	157,447
13-S-4	CHAGRIN FALLS	——	STW	209,000
14-S-1	CITY OF CLEVELAND	1,035	STW & INTER	18,734,435

STW - SEWAGE TREATMENT WORKS
PS - PUMP STATION
FM - FORCE MAIN

LEGEND

AREA NO. NATURAL DRAINAGE & SEWERAGE AREA DESIGNATION
EXISTING SEWAGE TREATMENT WORKS
PROPOSED ENLARGEMENT OF EXISTING SEWAGE TREATMENT WORKS
PROPOSED SEWAGE TREATMENT WORKS
◎ PROPOSED SEWAGE PUMPING STATION
PROPOSED FORCE MAIN
EXISTING SEWAGE INTERCEPTOR & MANHOLES
PORTION OF IMMEDIATE PROJECTS WHICH MIGHT BE CONSTRUCTED BY A REGIONAL DISTRICT
PORTION OF IMMEDIATE PROJECTS WHICH MIGHT BE CONSTRUCTED BY LOCAL MUNICIPALITIES
PROJECT PROPOSED FOR FUTURE CONSTRUCTION

REGIONAL PLANNING COMMISSION
CLEVELAND - CUYAHOGA COUNTY - OHIO
SEWER AND WATER PLAN
SANITARY SEWERAGE & STORM DRAINAGE
SANITARY SEWERAGE
SANITARY PROJECTS PROPOSED BY AREA ENGINEERS FOR IMMEDIATE CONSTRUCTION & MAIN INTERCEPTORS WHICH MIGHT BE BUILT BY A REGIONAL DISTRICT

FIG. 1 (CONTINUED)

Thirty-four of these projects, costing an estimated $30,846,000, are considered ready for immediate construction.

CREATION OF A UNIFIED AGENCY

Under the terms of the resolution initially employing Albright and Friel Inc., the Engineers were requested to prepare a program for the construction, financing, and administration of the necessary physical works and structures in the program. It is concluded that to solve the problem satisfactorily, it is necessary that there be created, or expanded in power, a political subdivision having jurisdiction for all of Cuyahoga County. This entity would be either a regional water and sewer district as permitted under the statutes of the State of Ohio, a County form of government organized under a charter containing powers at least equivalent to that granted to regional water and sewer districts, or some other governmental agency of equivalent powers.

Throughout this paper reference will be made to a Regional District primarily for convenience; it is contemplated that such terms include a county with a charter granting adequate powers or some other sufficiently empowered subdivision.

PAYMENT FOR TRANSFER OF EXISTING FACILITIES

Any successful program must contemplate that the District would acquire all sewage treatment works, together with their main intercepting sewers. In such acquisition, it is contemplated that the District would acquire such facilities by an agreement to pay the outstanding remaining debt on such sewage treatment works and main intercepting sewers. It would acquire all of the other existing facilities. Since they would not be of a regional character, under the definition established, they should be acquired without cost to the District. The remaining debt on these facilities is nothing more than an assessment of the remaining benefits for the local area served.

However, in the case of the City of Cleveland, two alternatives are suggested. Under the plan for the payment of outstanding debt, the District would pay the yearly debt service charges of the City of Cleveland, for sewage treatment works and main intercepting sewers, which would be about an average of $561,500 a year for the next 22 yr.

An alternative to this method of settlement has been developed. The City, at the present time (1960), collects a sewer rental of 25¢ per 1,000 cu ft from city residents, and 46¢ per 1,000 cu ft from suburban sewer users. This differential was developed by an Advisory Board to the City in 1939, and represents the City user's share of fixed charges which are and have been paid for out of ad valorem taxes. Under the alternative, the District would pay the City of Cleveland a yearly sum equal to the product of the amount of water billed for sewage charges within the City of Cleveland, multiplied by the differential in rate between the city users and suburban users. If this premise is carried out, there would be a yearly payment of $1,333,500.

For the purposes of an exploratory study of a required rate structure, the yearly sum of $1,333,500 has been used as an obligation of the proposed District to the City of Cleveland which would be carried over 22 yr.

Variations on the discharging of this obligation would be: (1) A direct yearly payment to the City, (2) a preferred sewer rental rate to the City of Cleveland users, (3) a combination of both.

It is contemplated that the proposed Unified Agency would take over and maintain all storm sewers, drainage ditches, and drainage structures which would have 1/2 sq mile or more of drainage area. These existing facilities would be taken over at no cost to the Agency.

DIVISION OF COST RESPONSIBILITIES ON NEW CONSTRUCTION

In regard to the cost responsibilities of new construction, the recommendations can be summarized as follows:

(a) That in sanitary sewer systems, branch sewers, and branch intercepting sewers would be constructed by the local municipality which could be financed by funds raised by ad valorem taxes or an assessment on a front foot and/or area basis.

(b) That main sanitary interceptor sewers would be constructed by the Unified Agency with funds raised to the extent of 90% of the cost by an assessment against the area served, and the other 10% would be paid through sewer rentals. The assessment bonds should mature over a 20-yr period and the assessment to properties benefited could be paid in twenty yearly installments. Any relief sewers constructed to supplement an existing main intercepting sewer would be financed entirely through sewer rentals.

(c) That sewage treatment plants would be financed by the Unified Agency with the issuance of bonds supported by sewer revenues, which bonds would mature over a 40-yr period at maturities established so that the yearly amortization cost of principal and interest would be approximately equal.

(d) That storm sewer projects, limited to projects with not less than 1/2 sq mile drainage area, would be financed and constructed by the Unified Agency which would be supported by ad valorem taxes paid by the entire country. Drainage structures on areas less than 1/2 sq mile in drainage area would be constructed by the local municipality.

ASSESSMENT AND REVENUE REQUIREMENT STUDIES-SANITARY PROJECTS

Assessment and revenue requirement studies have been made which entailed the preliminary assembly of factual data on tributary areas to proposed main intercepting sewers, population now served and to be served by the sewer system, operating and maintenance cost of existing treatment plants and sewers, together with an estimation of costs that would be involved in the administration and engineering force required for a Unified Agency.

Intercepting Sewer Assessment Charges.—In the development of assessment studies, as previously stated, it was contemplated that main intercepting sewers would be assessed against the properties benefited on an acreage basis. The results of these exploratory studies indicate that for the seventeen projects involving main intercepting sewers now under consideration, the assessment costs per net acre would range from a low of $35 per acre to a high of $1,095 per acre, with a median of $320 per acre. The assessment per house, using 2.4 houses per acre would range from a high of $460 a house to a low of $15 a house, with a median of $134 a house. These assessments could be paid by the property owners in twenty equal annual installments and would have the following range:

	Yearly Assessment Charge
High	$32 per yr
Median	9 per yr
Low	1 per yr

Sewer Rental Charges.—Sewer rentals must recover sufficient revenues to pay for a small portion of the amortization cost of intercepting sewers, the amortization cost of treatment plants, payments for existing facilities, operation and maintenance costs of sewers and sewage treatment works. Projected sewer revenues have been developed under two alternate plans as follows:

Plan No. 1 - A District embracing all of Cuyahoga County, including the City of Cleveland.
Plan No. 2 - A District embracing all of Cuyahoga County, excluding the City of Cleveland.

Plan No. 1 has been further subdivided as follows:

A) Based on a yearly payment for existing facilities to the City of Cleveland, without a preferential sewer rental rate.

B) Based on a yearly payment to the City of Cleveland, with a partial preferential sewer rental rate.

C) Based on no payment to the City of Cleveland, with a full preferential sewer rental rate.

A tabulation of the sewer rentals under the subdivisions in Plan No. 1 are summarized as follows:

	"A"	"B"	"C"
Yearly Payment to City of Cleveland	$1,333,500	$776,250	0
Rate to Cleveland Users			
Per 1,000 Cu Ft Per	$0.69	$0.60	$0.48
Equivalent Connection (15,000 Cu Ft/Yr)	$10.35	$9.00	$7.20
Rate to Others in Cuyahoga County			
Per 1,000 Cu Ft	$0.69	$0.69	$0.69
Per Equivalent Connection	$10.35	$10.35	$10.35

Under Plan No. 2, exploratory sewer rental charges to all participants in the Unified Agency have been developed based on two subdivisions. These subdivisions are on the basis of the present charge for sewage treatment by the City of Cleveland, of 46¢ per 1,000 cu ft and a possible future charge by the City to show the reflection of an increase which, for purposes of preliminary discussion, has been shown as 52¢ per 1,000 cu ft. The estimated sewer rentals to all participants of the District, together with the estimated yearly charge to be paid to the City for sewage treatment, are summarized as follows:

Subdivision	Rate Charged for Sewage Treatment by Cleveland Per 1,000 Cu Ft	Yearly Payment to Cleveland	Charge to Users Per 1,000 Cu Ft	Charge to Users Per Equivalent Connection Per Year
A	$0.46	$657,400.00	$1.15	$17.20
B	$0.52	$743,100.00	$1.19	$17.70

STORM SEWER CHARGES

The amount of general obligation bonds required to defray the cost of immediate storm sewer improvements would be $30,846,000. Based on a 20-yr issue of general obligation bonds bearing interest at 3-1/2%, and based on assessed valuation of $4,700,748,044 in the county, the yearly debt service requirements and the tax increase would be as follows:

	Yearly Debt Service Requirement	Estimated Tax Increase Per $100 Assessed Value	Estimated Tax Increase For House With $10,000 Assessed Value
First Year	$2,536,000	$0.0537	$5.37
Average for 20 Yr	$2,050,000	$0.0426	$4.26

ADVANTAGES AND DISADVANTAGES OF THE UNIFIED PLAN

Morris M. Cohn in an article in the October, 1957 issue of "Wastes Engineering," stated:

> "The era of metropolitan growth which is spreading like wildfire from coast to coast poses a great challenge to the sewage and industrial wastes field."—and went on further to say: "This is the time for a cosmopolitan approach to a cosmopolitan problem. The parent city cannot turn a deaf ear to the needs of suburban areas which surround the city boundaries, nor can the fringe areas use the attitude of a child which leaves the family circle and expects and demands support from its parents. There is an integration of interests which should serve as an amalgamating force to fuse the municipality and its ring areas into a stronger and more useful governmental alloy. There exists a two-way challenge to the parent community and to the fringe area, if common utility services could be of common good to both."

This article illustrates the fact that the sewage and drainage problems in the Cleveland-Cuyahoga County Area are similar to the problems faced by other areas throughout the country, and, also, illustrates the spirit of cooperation that has existed in Cuyahoga County in the past and that must continue in the future if common problems are to be solved.

The formation of a Unified Agency to acquire, construct, and maintain the Sanitary Sewer Systems and Storm Sewer Systems in Cuyahoga County, to-

TABLE 1.—ANNUAL SEWER RENTAL CHARGES IN CITIES OF OVER 100,000 POPULATION FOR DOMESTIC SEWER USERS DISCHARGING 15,000 CU FT OR 112,200 GAL OF WASTE PER YR

Philadelphia, Pa.	$27.66
Little Rock, Ark.	27.72
Camden, N.J.	33.68
Jersey City, N.J.	32.30
Chattanooga, Tenn.	14.04
Chattanooga, Tenn. - Suburban	21.36
Knoxville, Tenn.	22.20
Knoxville, Tenn. - Suburban	33.60
Akron, Ohio	16.50
Akron, Ohio - Suburban	18.15
Canton, Ohio	12.60
Canton, Ohio - Suburban	19.70
Columbus, Ohio	20.25
Columbus, Ohio - Suburban	33.75
Dayton, Ohio	8.40
Dayton, Ohio - Suburban	10.50
Toledo, Ohio	12.90
Toledo, Ohio - Suburban	18.15
Youngstown, Ohio	41.70
Youngstown, Ohio - Suburban	52.80
Cincinnati, Ohio	12.00
Cincinnati, Ohio - Suburban	12.00
Cleveland - Present with Tax Subsidy	3.75
Cleveland - Suburban Present	6.90
Proposed Under Plan 1	
Cleveland - Proposed with Tax Subsidy as at present	7.20
with no Subsidy	10.35
Cleveland - Suburban - Proposed	10.35
Proposed Under Plan 2	
Cleveland - Suburban - Proposed	17.70

gether with a central administrative control of operation of the sewage plants, would answer this two-way challenge in the Cleveland-Cuyahoga County Area, and would have the following advantages:

1. The water supply of the region which is primarily represented by the City of Cleveland Water System would be protected from the ever increasing discharge of overflowing cesspools and inadequately treated sewage.
2. The rivers and streams in the area would be preserved for recreational use.
3. The enlarged agency would be in a position to provide treatment facilities and intercepting sewers on a drainage area basis which is the logical method of division instead of the division by political boundaries.
4. A Unified Agency, with appropriate powers, could facilitate and, in some cases, make possible the financing of necessary sewers and sewage treatment works which might be impossible to finance by the individual municipalities with today's high construction cost and high interest cost on bond issues. While at the present time some municipalities are being assisted by the County through financing under County backed bonds, this assistance cannot cover all projects which now require attention.
5. The solving of drainage problems on a drainage area basis, in lieu of a political boundary basis will allow for an orderly development of necessary drainage facilities and the protection of properties along the streams and rivers.
6. The engineering staff of the District would be in a position to make exploration studies and preliminary designs for storm sewers and sanitary sewers in needy areas, anticipating necessary improvements.

The disadvantage of the Unified Plan would be the increased cost which the residents of the City of Cleveland would have to pay for sewage treatment. This increase in cost can be established by the difference between (1) the actual cost for sewage treatment when the City of Cleveland plants would be expanded and (2) rental cost to the City residents under Plan One. The charge to residents and non-residents of the City of Cleveland would be approximately 52¢ per 1,000 cu ft or $7.80 a yr per 15,000 cu ft. This would compare with a charge under Plan No. 1 of $10.35 per 15,000 cu ft, or a difference of $2.55 per yr per average domestic connection. The sewage treatment charge, as determined herein for the average domestic connection discharging 15,000 cu ft per yr, can be compared with the charges made for a like quantity of sewage in cities of 100,000 population or larger as shown in Table 1. The proposed rate in Cleveland is less than in any of these cities except Dayton.

This report cannot be expected to produce a fixed formula which will instantaneously be acceptable to officials of all municipalities concerned. The report, however, does expose the various facets of the problem so that men and women with a common purpose can intelligently negotiate agreements which will solve this difficult problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3133

SEDIMENTATION IN RESERVOIRS IN THE SOUTHEAST

By John E. Jenkins[1], Charles E. Moak[2], and Daniel A. Okun[3], F. ASCE

SYNOPSIS

The most recent reservoir sediment data have been analyzed in this report to evaluate factors that can be used to predict sedimentation in reservoirs in the southeastern United States. The most significant result of this study is that in the southeastern part of the United States, the annual volume of sediment is directly proportional to drainage area, averaging 0.44 acre-ft. per sq. mile.

INTRODUCTION

The depletion of storage in reservoirs due to the deposition of silt is an economic problem that cannot be ignored in the design of such structures. Studies made in the 1930's by the Soil Conservation Service of the United States Department of Agriculture (1a)[4] indicate that about 64% of all the reservoirs on silt-carrying streams in the United States have useful lives of less than 100 yr. This estimate was based on a sample of about 1% of the reservoirs in existence at that time. It was assumed that a reduction of 80% in the capacity of a reservoir terminates the useful life of the structure.

Note.—Published essentially as printed here, in July, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2557. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[4] Numerals in parentheses refer to corresponding items in Appendix II.

This paper reviews the origin of silt, its mode of transportation and deposition, factors that affect the rate of sedimentation, its economic effects, and measures of control. It also includes an analysis of the available data on reservoir sedimentation in the southeastern United States. Most of the data have been taken from Sedimentation Bulletin Number Five (2), which is a summarized presentation of the data from all known, reliable, sedimentation surveys made in the United States, through 1950.

These data, to the writers' knowledge, have not been subjected to statistical analysis for the purpose of defining the relationships between some of the measurable factors affecting the rate of sediment production and reservoir silting. An attempt is made in this paper to correlate the rate of sedimentation in a reservoir with the size of the drainage area. An attempt is made to correlate the annual sediment volume with the reservoir capacity-watershed ratio.

In addition, an indication is made of the direction that further analysis should take when more data are available.

HISTORY

Interest in siltation of reservoirs, sediment information and the practice of making observations of sediment loads has been growing steadily in nearly all countries of the world since the turn of the century.

The earliest measurements of sediment on record were made in the Rhone River in 1808, and 1809 (3). Blume made other early observations in the Elbe River at Harburg, Germany, from 1837 to 1854, and Baumgarten in the Garonne River in southwestern France from 1839 to 1846.

Records show that the earliest sediment measurements in the United States were those made by Talcott in the Mississippi River, in 1838. Considerable study was made in this river, as well as in the Missouri River, in the latter half of the 19th century, and also in many streams in the southwestern part of the United States. These studies in the southwest were instigated by irrigation, the development of which was made more difficult due to the heavy sediment loads.

Reservoir sedimentation studies became numerous in this country in the depression years of the early 1930's. The Soil Conservation Service, in 1934, began a general nationwide investigation of silt accumulations in selected impoundments. By 1939, a total of 76 surveys had been made; and by 1950, data had been accumulated on several hundred reservoirs. This information has been compiled under the auspices of the Subcommittee on Sedimentation, Federal Inter-Agency River Basin Committee. This Subcommittee is composed of members representing the Department of Agriculture, Department of Commerce, Department of the Interior, Department of Health, Education and Welfare, Federal Power Commission and the Tennessee Valley Authority.

FACTORS AFFECTING THE FORMATION, TRANSPORTATION, AND DEPOSITION OF SILT

Silt may be defined as mud, fine earth, or rock that is transported by a stream of water. It is the result of soil erosion on the watershed and stream banks. The impact of raindrops, turbulent overland flow, and the dynamic action of water in channels contribute to the total silt load of a stream.

Sheet erosion generally constitutes the major portion of the silt load in streams in humid regions (4), while bank erosion may be significant in arid or semi-arid regions.

TABLE 1.—SUMMARY OF RESERVOIR SEDIMENTATION STUDIES IN THE SOUTHEASTERN UNITED STATES

Reservoir	Stream	Nearest Town	Drainage Area in Square Miles		Initial Capacity Acre-ft	Initial Capacity Water-Shed Ratio Acre-ft/sq mi	Date of Survey	Length of Record[a] yrs	Av Annual Sediment Accumul.		Av Annual Storage-Loss, Percent	Total Percent Loss to Date of Survey	Av Annual Runoff Acre-ft
			Total	Net					Acre-ft	Acre-ft /sq mi			
North Carolina													
Lake Apex	Swift Creek	Apex	4.0	4.0	106	26.5	June 41	16.0	0.76	0.19	0.71	11.3	
Franklinton	Sallie-Keaney Cr.	Franklinton	1.13	1.12	34.7	30.7	May 38	13.3	0.58	0.509	1.60	21.33	
Lake Brandt	Reedy Fork	Greensboro	74.1	73.4	2870	38.7	Aug 34	11.5	22.8	0.308	0.79	9.06	
Sanford City	Lick Creek	Sanford	3.75	3.73	103	27.5	June 41	14.33	0.56	0.150	0.54	7.77	
Burlington Municipal	Stoney Creek	Burlington	105.2	105.0	1488	14.1	Sept. 49	21.3	13.5	0.128	0.90	19.22	
Walnut Cove	Dan River	Walnut Cove	397	397	974	2	Apr 32	9.0	85.0	0.214	8.73	78.5	357,000
Cannon Lake	Buffalo Creek	Kannapolis	18.0	17.7	2600	144	June 41	1.9	13.9	0.774	0.53	1.00	
Lake Concord	Chambers & Rose Dr.	Kannapolis	4.7	4.54	1201	256	May 35	10.2	8.04	1.71	0.65	6.58	
Eury	Little River	Troy	269	269	1104	4.10	Mar 40	25.0	5.10	0.019	0.46	11.6	
Pee Dee Mfg. Co.	Hitchcock Creek	Rockingham	176	25	464	2.64	Mar 40	66	6.34	0.036	0.20	12.93	
Norwood Lake	Pee Dee River	Mt. Gilead	4600	431	136,833	30.0	Mar 40	11.75	3200	0.696	0.22	2.57	
Cheoah	Little Tenn. River	Tapoco	1608	1607	35,030	27	Aug 41	22.7	396	0.246	0.90	20.441	
Chatuge	Hiwassee River	Hayesville	189	178	247,800	1311	Aug 49	7.5	59.5	0.315	0.023	0.169	
Fontana	Little Tenn. River	Fontana	1571	1426	1,444,300	913	Mar 50	5.4	655	0.417	0.041	0.222	3,000,312
South Carolina													
Lancaster	Turkey Qtr. Creek	Lancaster	9.40	9.34	242	25.7	June 38	13.4	3.92	0.417	1.61	21.49	
Spartanburg Municipal	S. Pacolet River	Fingerville	91.33	90.8	3506	38.4	Mar 47	20.9	29.6	0.324	0.84	17.54	
Georgia													
Nottely	Nottely River	Blairsville	214	207	184,400	862	Aug 49	7.6	133	0.623	0.07	0.533	289,133
Sequoyah	Small Branches	Jasper	1.60	1.51	890	556	July 39	10.0	2.65	1.66	0.28	2.81	
White Manganese No. 6	Pettit Creek	Cartersville	12.46	11.0	1021	81.9	Nov 38	9.2	14.9	1.20	1.29	11.85	
Alabama													
Lay	Coosa River	Clanton	9087	9076.5	156,525	17.2	May 36	22.3	8.08	0.089	0.52	11.50	11,240,300
Lake Purdy	Little Cahaba River	Birmingham	41.74	40.22	19,080	457	Nov 35	25.2	20.0	0.479	0.10	2.55	
Guntersville	Tennessee River	Guntersville	24,450	2550	1,018,700	41.6	June 47	8.4	31,500	1.280	0.32	2.692	25,226,338
Wilson	Tennessee River	Florence	30,750	1135	562,500	18.3	Dec 36	12.6	25,600	0.833	1.13	14.292	36,055,995
Mississippi													
Arkabutla	Coldwater River	Arkabutla	1,000	948	525,300	525	Dec 47	6.3	630	0.630	0.114	0.72	521,400
Tennessee													
Pickwick Landing	Tennessee River	Pickwick	32,820	1997	1,091,400	33.3	Sept 46	8.6	25,600	0.780	0.143	1.227	33,252,195
Cherokee	Holston River	Jefferson Cy	3429	3381	1,559,570	455	Apr 54	12.4	850	0.248	0.061	0.756	3,076,393
Douglas	French Broad River	Sevierville	4541	2854	1,514,100	334	July 49	6.4	3860	0.850	0.16	1.025	4,800,601
Norris	Clinch River	Norris	2912	2823	2,567,000	702	June 46	10.3	1090	0.376	0.052	0.535	2,656,901
Ocoee No. 3	Ocoee River	Ducktown	496	263	12,002	24.2	July 45	2.9	1130	2.293	4.22	16.093	630,693
Ocoee No. 3	Ocoee River	Ducktown	496	263	11,255		Nov 46	4.2	1120	2.259	4.15	21.316	780,019
Ocoee No. 3	Ocoee River	Ducktown	496	263	10,391		Aug 48	6.0	1060	2.129	3.92	27.356	758,795
Ocoee No. 3	Ocoee River	Ducktown	496	263	9849		Aug 50	8.0	920	1.856	3.41	31.145	810,455
Ocoee No. 1	Ocoee River	Parksville	595	96	91,300	153.5	Sept 49	37.7	3900	6.55	0.55	25.971	981,662
Hales Bar	Tennessee River	Jasper	21,790	990	160,850	7.41	Oct 30	17.0	33,600	1.541	0.95	16.137	27,611,594
Hales Bar	Tennessee River	Jasper	21,790	990	154,200		Oct 35	22.0	33,100	1.520	0.89	19.604	26,811,325
Hales Bar	Tennessee River	Jasper	21,790	990	154,084		Oct 40	27.0	25,800	1.185	0.73	19.664	26,386,179
Hales Bar	Tennessee River	Jasper	21,790	990	153,045		July 47	33.7	21,400	0.975	0.6	20.206	25,600,526

[a] From date of reservoir construction to date of survey. Reservoirs included in the study but not in the above summary are the following: Highpoint, Lake Michie, University Lake, Roxboro City Lake, Albermarle City Lake, Entwhistle No. 3, High Rock, Lake Lee, Salem, Lexington, Hiwassee, Chester, Appalachie, Lake Isaqueena, Lloyd Shoals, Newman, Lake Auburn, Bayview, Lake Harris, Wheeler, Radnor Lake, Malichucky, Garyville, Apalachie, Chickamauga.

Witzig (5a) suggests that once material has been eroded, movement to a reservoir may be as suspended load (for the material in suspension) and as bed load. The bed load may be considered to consist of particle sizes too large for movement as part of the suspended load. These particles are transported by a rolling action along the bottom of the stream. The limiting particle size or the "dividing" grain size is a function of the discharge and slope of the stream. Particles coarser than this "dividing" size move at a rate proportional to stream discharge and slope.

The turbulence theory offers an explanation of suspended sediment transportation. According to this theory, there is a random, irregular transfer of energy from the fluid to silt, and from silt particle to silt particle. The energy transferred is the difference between forces that buoy the particles and those that tend to remove them. Gravity, the drag force of the water, and eddy currents are the important factors.

Regardless of the mode of transportation of debris to the reservoir, the pattern of deposition is essentially the same for all impoundments. When the fast moving, silt-laden water meets the quiet water of the reservoir, the influent flow loses velocity rapidly, and the heavier particles settle out to form a delta in the mouth of the stream (6). The lighter particles are distributed farther out in the reservoir or are not deposited at all.

Density currents also affect the distribution and accumulation of silt. The silt-bearing water has a greater density than does the water in the reservoir and tends to underrun the impoundment rather than to mix with that already present. This phenomenon is accentuated when the temperature of the inflowing water is somewhat less than that of the impounded water because the density of water varies inversely with its temperature.

Grover and Howard (7) reported that this phenomenon was observed three times on Lake Mead, in 1935. Muddy water was observed leaving the reservoir through flood gates while the surface water was relatively clear. This offers an explanation as to why deposits are usually greater in the stream channel throughout the length of the impoundment rather than uniformly distributed over the bottom. This also may explain why, many times, sizeable deposits are found in deep areas immediately above the dam. Important silt control benefits may accrue when the action of these currents are more fully understood.

Silt composed of fine earth or rock can further be defined in terms of its size. The American Society for Testing Materials (ASTM) has classified silt as particles ranging from 0.005 mm to 0.05 mm, the Massachusetts Institute of Technology (MIT) from 0.002 mm to 0.06 mm, the Bureau of Soils, United States Department of Agriculture from 0.005 mm to 0.05 mm, and the National Park Service, United States Department of the Interior from 0.006 mm to 0.2 mm (8).

SILT CONTROL MEASURES

The process of erosion is as old as the earth itself. Man has only accelerated the process by destroying the natural balance between resistance to erosion (vegetative cover) and erosion attack (rainfall and runoff). The solution to the problem of control then may take one of two directions: 1) removal of silt deposits from the reservoir, and 2) control of silt formation, entrance, and deposition.

Generally, it is not feasible to remove sediment by mechanical means. Dredging (hydraulically or mechanically) has been employed effectively only on narrow, channel-type impoundments.

Sluicing is another procedure employed for desilting work. This involves the opening of large gates near the base of the dam to either flush out deposits or to waste silt-laden flood waters that are underrunning the reservoir. One disadvantage with this procedure is obvious if flood control is the primary function of the impoundment. Also, sluicing usually is effective in removing only those deposits in the stream channel immediately above the dam.

Perhaps the second mentioned direction, that of control of erosion and deposition, offers the most promise. Vegetative screens and engineering structures may be used to cause deposition in the valley prior to entrance into the reservoir. Aggradation in the upstream channels is feasible only when such areas are worthless. Taylor (14a) describes the benefits that accrued from the accidental introduction of tamarisk, an evergreen shrub, to the upper end of Lake McMillan in New Mexico. This vegetative screen is credited with reducing the annual rate of silting from 1930 acre-ft to 350 acre-ft. The presence of such a screen shifts the location of the deposits from inside the impoundment to the valley above.

Erosion control seems the most promising answer to the problem. Good soil conservation practices not only keep much silt out of the impoundment, but these practices also keep the sediment on the ground where it is useful. This "upstream engineering" as it has been termed (5) consists of reforestation, contour plowing, crop rotation, and proper treatment of such things as unstabilized roadway fill, high embankments, and stream banks.

Eakin (1b) states that

> "Erosion control not only has the effect of conserving lands in the drainage area, but is outstanding as one of the fundamental and permanently practicable means of reducing the rate of reservoir silting. It inhibits primary production of debris and, thus, involves no progressive and ultimately embarrassing accumulations above, or troublesome sediment-laden discharge from, the reservoir."

SILT AS AN ECONOMIC PROBLEM

More reservoirs have been built in the United States since 1925 than in any other period of history. As stated previously, the life expectancy of many of these are dangerously short.

Brown (9a) estimated in the early 1940's that the direct annual loss in investment due to reservoir silting was at least $10,000,000 and was probably much more. In many cases, a reservoir has become completely filled with silt before its cost has been amortized.

It must always be kept in mind that the economic problem is twofold. First, there is loss of topsoil on the watershed that results in poorer land, and secondly, this same topsoil or silt ultimately occupies space constructed for some other purpose.

The economic life of a reservoir is a function of a number of factors. One of the most important of these is the nature of the use of the impounded water (10). Other factors involved are;

(1) the effectiveness and economic justification of possible control measures,
(2) the replacement cost of the existing reservoir, and
(3) the effects of sedimentation on wildlife, fish, scenery and navigation.

The actual cost of reservoir sedimentation is difficult to determine, and computed values will vary depending on the basis used for depreciation. Original construction costs are not a sound basis for computing losses because of the variation of construction costs over a period of time. These can be adjusted to the present time by use of indices such as those published by the Engineering News Record.

If actual replacement of storage must be provided, it should be remembered that, generally, the most economical site is the one selected for the first structure. Adding to a structure or constructing a dam at a new site will usually cost considerably more than the first structure.

In the ultimate computation of damages occasioned by sedimentation, any one of three standard methods may be used:

(1) The annual cost method,
(2) the present worth method, and
(3) the capitalized cost method, which is a variation of the present worth approach.

The annual cost method is more generally used by engineers, and is probably more readily understood by business-men who are accustomed to thinking in terms of annual expenditures.

Blench (11) raises the question as to whether losses due to siltation are true ones since losses can only by caused by forces unforseen or forces whose occurrence is a matter of probability. Since siltation is a known factor whose rate can be approximated, "losses" can be provided for in design.

Regardless of terminology siltation is, and will remain, a problem, and the decrease of reservoir capacity due to this must be taken into consideration in some manner. Studies of sedimentation in Lake Mead (12) have produced estimations that it will be completely filled in 2380 AD, 445 yr after completion of the dam. Though the rate of siltation here is less than in most reservoirs, it is still a problem and will have to be dealt with at some future date.

Studies of sedimentation in the Burlington, North Carolina Municipal Reservoir (13) show that with even a minimal (of the reservoirs studied in this report) annual sedimentation accumulation of 0.128 acre-ft. per sq. mile, siltation can be an expensive item. The Burlington Reservoir was built in June, 1928 at a cost of $318,000. Based on the Engineering News Record Cost Index of 1953, the present cost of this same structure would be approximately $858,000. 19.22% of the original capacity was filled with sediment between the date of construction and the last sedimentation survey in September, 1949, an interval of 21.3 yr. On a straight-line depreciation basis, the annual cost of siltation at this rate is about $8,000 based only on the original investment.

Few generalizations can be made pertaining to the economic effects of reservoir sedimentation. While each reservoir must be analyzed in the light of applicable data, the cost figures cited above do indicate something of the economic burden imposed by silt.

PREVIOUS STUDIES OF RATES OF SEDIMENTATION

The rate at which silt accumulates in a reservoir is a function of many independent and interrelated variables. Some of these factors may be listed as follows:

1. The area and topography of the watershed,
2. the character of the soil and vegetation on the catchment area,
3. the rate and amount of runoff,

4. the rate and amount of rainfall,
5. shape of the impoundment,
6. the ratio of the reservoir capacity to the watershed area, and
7. the method of reservoir operation.

Only general statements can be made concerning the effect of these variables on reservoir sedimentation. No one factor seems to be the most important in every case. However, cultivated land on steep slopes always contributes greatly to the rate. It also appears that the rate of rainfall is more important than the amount because silt production increases as the intensity of the rainfall increases. Though a wet watershed produces a greater runoff, silting per unit time decreases as the duration of a constant rainfall increases.

Unfortunately, there are not sufficient data available to determine the extent to which all the variables affect the rate of silt accumulation. However, the reservoir storage capacity, silt accumulation, and size of the drainage area are usually determined in most sedimentation surveys. Many of the attempts to formulate rate relationships have been based on these factors.

Witzig (5b) has attempted to correlate the reservoir capacity-watershed ratio to annual sediment accumulation by making a logarithmic plot of these variables from data available in the early 1940's. By drawing envelope curves "by eye" to bound the data representing several geographical regions, he obtained a generalized equation of the following form:

$$\Delta S_R = I\,(S_R)^{0.83} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which ΔS_R is the annual silting rate in acre-ft. per sq mile of drainage area, I denotes the coefficient, termed "regional index," and S_R refers to the original storage in acre-ft per sq mile of drainage area.

His regional index, based on data from 19 reservoirs in the southeastern States of Alabama, Georgia, Virginia, Maryland, and North and South Carolina, ranges from a lower limit of 0.00307 to an upper limit of 0.0375. According to this relationship, the rate of sediment accumulation may differ between two reservoirs of similar capacity and watershed area, approximately, by a factor of ten.

Brown (9b), in commenting on Witzig's work, states that the equation presented is not valid at the limits since the upper limit of sedimentation is a function of the amount of silt brought in by a stream rather than the capacity of a reservoir. Instead, he suggests a plot of annual sediment accumulation per sq mile of drainage area versus total drainage area. He has drawn envelope curves to bound data from about 30 reservoirs in the Southern Piedmont (states not indicated). The mean curve drawn "by eye" indicates that the rate of silting decreases as reservoir size increases. The plot also indicates that a limiting rate of sedimentation is approached with increasing watershed area.

EVALUATION OF SEDIMENTATION RATES

The parameters suggested by both Witzig and Brown have been studied by the writers in the light of additional data collected from a larger area of the southeast. For this study, data from 56 reservoirs in the southeastern states of North and South Carolina, Georgia, Alabama, Mississippi, and Tennessee have been analyzed by statistical procedures. These data represent all the known, reliable sedimentation surveys (2) made in these states through 1950.

Fig. 1 is a graphic presentation of these data according to the parameters suggested by Witzig. Instead of drawing envelope curves to bound the data, the equation of the straight line of best fit has been computed by the method of least squares. This regression line and the 95% confidence limits for the dependent variable (S_R) are shown on the figure. The equation may be stated

$$\Delta S_R = 0.127\, S_R^{0.308} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The correlation coefficient (r) for these variables is found to be 0.383. This low coefficient of correlation indicates little since correlation is generally assumed doubtful at values less than 0.5.

A more significant test (16) is available by transforming the distribution of the correlation coefficient (r) by

$$Z = \log_e \frac{1 + r}{1 - r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Assuming no correlation, the variable (Z) is approximately normally distributed with variance.

$$\frac{1}{(n - 3)^{\frac{1}{2}}}$$

(in which n is the number of observations). The standardized normal variate $\left(\frac{Z}{\theta}\right)$, when using the correlation coefficient of 0.383, is 2.94. The critical value of the standardized normal variate at the 5% significance level is 1.96. Therefore, it can be accepted at this level that correlation exists since the value obtained from the data exceeds this critical value. A value as high as 2.94 would lead to acceptance of the assumption of no correlation only for a significance level less than 0.37%.

Having established that correlation does exist, a test for the degree of correlation existing can be made by squaring the correlation coefficient. When applying this test to an r of 0.383, it is found that only 15% of the variation of the dependent variable (ΔS_R) can be accounted for by the variation of the independent variable (S_R). With such poor correlation, the primary usefulness of a plot such as this is to indicate the variation in sediment accumulation rates for reservoirs of a given size.

As a follow-up of Brown's work, the writers have plotted annual sediment accumulation versus total drainage area using more complete data[5] for the southeast. Fig. 2 shows the line of best fit of the data as determined by the least squares method as well as the 95% confidence intervals of the dependent variable (ΔS_R). This line, for all practical purposes, is horizontal (the slope is approximately zero on a logarithmic plot) and is defined by the following equation:

$$\Delta S_R = I\,(A_D)^0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which A_D denotes the total drainage area in square miles, and I is the ordinate intercept value.

According to this equation, ΔS_R has no relation to the size of the drainage area as

$$\Delta S_R = I\,(A_D)^0 = I = 0.44 \text{ acre-ft. per sq. mile.}$$

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[5] Table 1 substitutes notations for the standard symbols to conform to the notation commonly used in statistical analysis.

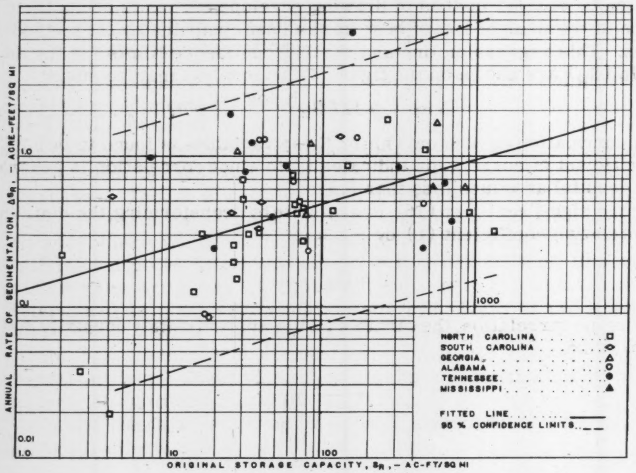

FIG. 1.—RELATION OF RATE OF SEDIMENTATION, ΔS_R, TO CAPACITY—WATERSHED AREA RATIO FOR RESERVOIRS IN THE SOUTHEASTERN UNITED STATES

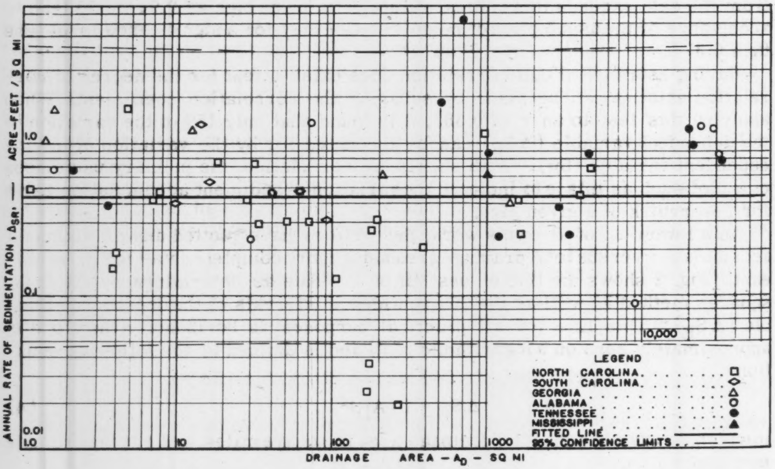

FIG. 2.—RELATION OF RATE OF SEDIMENTATION, Δ_{SR}, TO SIZE OF DRAINAGE AREA, A_D, IN THE SOUTHEASTERN UNITED STATES

TABLE 2.—DATA FOR FIG. 2

Reservoir	x_o	y_o	y_o'	X	X^2	Y	Y^2	$X - \overline{X}$	$(X - \overline{X})^2$	$(X - \overline{X})Y$
6-1	4.0	0.19	19.0	0.602	0.362	1.278	1.633	-1.613	2.602	-2.061
6-2	1.13	0.509	50.9	0.053	0.003	1.706	2.910	-2.162	4.674	-3.688
6-3	74.1	0.308	30.8	1.870	3.497	1.488	2.214	-0.345	0.119	-0.513
6-4	62.8	0.494	49.4	1.798	3.233	1.694	2.887	-0.417	0.174	-0.706
6-5	167.5	0.271	27.1	2.224	4.946	1.433	2.053	+0.009	0.000	+0.013
6-6	3.75	0.150	15.0	0.574	0.329	1.176	1.383	-1.641	2.693	-1.930
6-7	30.60	0.728	72.8	1.486	2.208	1.862	3.467	-0.729	0.531	-1.357
6-8	7.62	0.499	49.9	0.882	0.778	1.698	2.883	-1.333	1.777	-2.263
6-9	105.2	0.128	12.8	2.023	4.093	1.107	1.225	-0.192	0.037	-0.213
6-10	397.0	0.214	21.4	2.598	6.750	1.330	1.769	+0.383	0.147	+0.509
7-5	33.0	0.302	30.2	1.518	2.304	1.480	2.190	-0.697	0.486	-1.032
7-6	18.0	0.774	77.4	1.255	1.575	1.888	3.565	-0.960	0.922	-1.812
7-7	4.7	1.710	171.0	0.672	0.452	2.233	4.986	-1.543	2.381	-3.446
7-8	168.0	0.024	2.4	2.225	4.951	0.380	0.144	+0.010	0.000	+0.004
7-9	269.0	0.019	1.9	2.428	5.895	0.279	0.078	+0.213	0.045	+0.059
7-10	3930.0	0.462	46.2	3.594	12.917	1.664	2.769	+1.379	1.902	+2.295
7-11	50.5	0.302	30.2	1.703	2.900	1.480	2.190	-0.512	0.262	-0.758
7-12	176.0	0.036	3.6	2.244	5.036	0.556	0.309	+0.029	0.001	+0.016
7-13	27.68	0.444	44.4	1.442	2.079	1.648	2.716	-0.773	0.598	-1.274
7-14	4600.0	0.696	69.6	3.663	13.418	1.843	3.397	+1.448	2.097	+2.669
7-15	6.75	0.414	41.4	0.829	0.687	1.617	2.615	-1.386	1.921	-2.241
20-11	1571.0	0.417	41.7	3.196	10.214	1.619	2.621	-0.981	0.962	-1.588
20-12	1608.0	0.246	24.6	3.206	10.278	1.390	1.932	+0.991	0.982	+1.377
20-16	189.0	0.315	31.5	2.276	5.180	1.498	2.244	+0.061	0.004	+0.091
20-18	968.0	1.105	110.5	2.986	8.916	2.044	4.178	+0.771	0.594	+1.576
7-1	16.05	0.550	55.0	1.205	1.452	1.740	3.028	-1.010	1.020	-1.757
7-2	9.40	0.417	41.7	0.972	0.945	1.620	2.624	-1.243	1.545	-2.014
7-3	91.33	0.324	32.4	1.960	3.842	1.511	2.283	-0.255	0.065	-0.234
7-4	63.0	0.480	48.0	1.799	3.236	1.681	2.826	-0.416	0.173	-0.699
8-1	14.02	1.330	133.0	1.146	1.313	2.124	4.511	-1.069	1.143	-2.271
20-17	214.0	0.623	62.3	2.330	5.429	1.794	3.218	+0.115	0.013	+0.021
8-2	1414.0	0.408	40.8	3.150	9.923	1.611	2.595	+0.935	0.874	+1.506
11-1	1.39	1.080	108.0	0.143	0.020	2.033	4.133	-2.072	4.293	-4.212
12-1	1.6	1.660	166.0	0.204	0.042	2.220	4.928	-2.011	4.044	-4.464
12-2	11.46	1.200	120.0	1.058	1.119	2.079	4.322	-1.157	1.339	-2.405
12-3	1.6	0.660	66.0	0.204	0.042	1.819	3.309	-2.011	4.044	-3.658
12-4	9087.0	0.089	8.9	3.958	15.666	0.949	0.901	+1.743	3.038	+1.654
12-5	41.74	0.479	47.9	1.619	2.621	1.680	2.822	-0.596	0.355	-1.001
13-1	72.3	1.340	134.0	1.859	3.456	2.127	4.524	-0.356	0.127	-0.757
13-2	30.0	0.237	23.7	1.478	2.184	1.375	1.891	-0.737	0.543	-1.013
18-4	24,450.0	1.280	128.0	4.389	19.263	2.107	4.439	+2.174	4.726	+4.581
18-5	25,590.0	1.255	125.5	4.471	19.990	2.098	4.402	+2.256	5.090	+4.933
18-6	30,750.0	0.833	83.3	4.488	20.142	1.921	3.690	+2.273	5.167	+4.366
18-1	2.1	0.660	66.0	0.322	0.104	1.820	3.312	-1.893	3.583	-3.445
18-7	32,820.0	0.780	78.0	4.516	20.394	1.892	3.580	+2.301	5.295	+4.353
20-5	3429.0	0.248	24.8	3.535	12.496	1.394	1.943	+1.320	1.742	+1.840
20-6	1183.0	0.245	24.5	3.073	9.443	1.389	1.929	+0.858	0.736	+1.192
20-7	4541.0	0.850	85.0	3.657	13.374	1.929	3.721	+1.442	2.079	+2.782
20-13	35.98	0.393	39.3	1.556	2.421	1.595	2.544	-0.659	0.434	-1.051
20-14	2912.0	0.052	5.2	3.463	11.992	0.716	0.513	+1.248	1.558	+0.894
20-19	1018.0	0.854	85.4	3.007	9.042	1.931	3.729	+0.729	0.627	+1.529
20-21	496.0	1.856	185.6	2.695	7.263	2.268	5.144	+0.480	0.230	+1.089
20-22	595.0	6.550	655.0	2.775	7.701	2.816	7.930	+0.560	0.314	+1.577
20-23	20,790.0	1.215	121.5	4.317	18.636	2.084	4.343	+2.102	4.418	+4.381
20-24	21,790.0	0.975	97.5	4.338	18.818	1.989	3.956	+2.123	4.507	+4.223
15-26	1000.0	0.630	63.0	3.000	9.000	1.799	3.236	+0.785	0.616	+1.412
				124.034	364.370	92.502	164.684		89.649	+0.055

$x_o = A_D$ = Total drainage area in square mile.

$y_o = \Delta S_R$ = Average sedimentation in acre feet per square mile.

$X = \text{Log } x_o$; $\overline{X}$ = Mean X; $y_o' = (y_o)(100)$; $Y_o = \text{Log } y_o$; $\overline{Y}$ = Mean Y.

Statistically, applying the same tests for correlation as were used previously, these variables have no correlation since the correlation coefficient is a function of the slope of the line. However, by definition

$$\Delta S_R = \frac{V}{A_D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which V is the annual volume of sediment accumulation in acre-ft.

Then

$$\Delta S_R = \frac{V}{A_D} = I$$

or

$$V = I\,(A_D) = 0.44\,A_D \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

This equation indicates that annual volume of sediment accumulation is directly proportional to the size of the catchment area. Fair and Geyer (15) describe the relationship of the above parameters for the southwestern United States as

$$V = c\,A_D^{0.77} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which c is a coefficient of deposition that varies from 0.43 to 4.8.

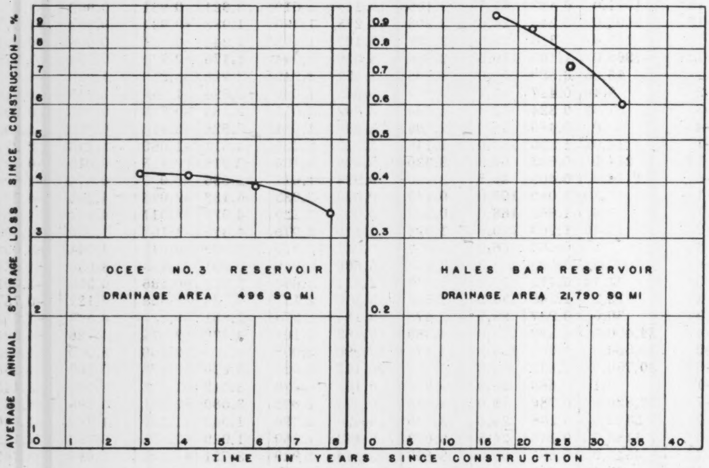

FIG. 3.—CHANGE IN ANNUAL PERCENTAGE STORAGE LOSS WITH TIME FOR TWO RESERVOIRS IN THE TENNESSEE RIVER VALLEY

Fig. 2 does not indicate any limiting sediment accumulation rate with increasing drainage area as Brown has indicated for a more limited geographic region. It appears, therefore, from the analysis of the data for the southeastern states, that both Eq. 2 and 6 are valuable only for generalizing the rate of sedimentation that may be expected.

Probably there is a much better correlation of both sets of variables within a limited geographic region such as the Southern Piedmont because many of the unevaluated factors such as land use, soil characteristics, slope of the watershed and runoff are relatively constant within such a region. However, the usefulness of such a relationship outside of such a region is questionable.

Another aspect of reservoir sedimentation that has not been fully examined is the reduction in annual percent loss of original storage capacity with time. In almost every instance, subsequent surveys on a given reservoir indicate a reduction in the silting rate. Fig. 3 is a graphic portrayal of this aspect for two Tennessee Valley impoundments. The reasons for this trend are not evident from the data available to the writers but the following explanations are suggested:

1. Initiation of effective soil conservation practices on the watershed progressively reduces siltation.
2. Decreased capacity in a reservoir causes a higher velocity of flow through the impoundment, which tends to keep the material in suspension.

Whatever the explanation of such a trend may be, evaluation of this phenomenon for a given reservoir can permit a more accurate estimate of the useful life of the reservoir to be made. Generally, the estimate is based on a constant rate of storage reduction.

Until the rate of reservoir sedimentation can be more accurately defined in terms of readily measured factors, provision for silt storage should be based on studies of accumulations in nearby reservoirs on watersheds of similar character.

SUMMARY AND CONCLUSIONS

Silt accumulation in a reservoir is a result of soil erosion on the watershed area. Factors believed to be important in the formation of silt are;

1. rate and amount of rainfall and runoff,
2. slope of the watershed,
3. land use on the catchment area, and
4. character of soil on the watershed.

Previous attempts to relate empirically sediment accumulation to the physical characteristics of reservoir capacity and size of drainage area within a geographic region have met with limited success. The writers have attempted to analyze, statistically, the data for the southeastern United States. This analysis indicates the following:

1. The annual volume of sediment formed is directly proportional to the drainage area. For the southeast, the annual sediment accumulation averages 0.44 acre-ft per sq mile of drainage area.
2. The annual rate of sediment accumulation per unit drainage area is related to the capacity-watershed ratio. However, statistical correlation is poor. This poor correlation precludes the use of the derived equations for design purposes.
3. The percent loss of original storage capacity decreases with time. Evaluation of this relationship for a given reservoir may lead to a better estimate of useful structure life.

It is apparent that attempts to relate the rate of sedimentation to reservoir capacity and watershed area leave much to be desired. It is suggested that further study include the following:

1. The relationship between sediment accumulation and the weighted average slope of the catchment area;

2. the effect of (a) average annual runoff and (b) annual peak floods on the rate of silt accumulation;
3. the relationship of reservoir surface loading, or overflow rate, to sedimentation since settling is a function of the surface loading; and
4. classification of reservoirs according to shape as it may affect deposition.

The need for more accurate methods of silt forecasting and control is apparent. The solution to this problem poses a challenge to those in the fields of soil conservation and water resources management.

ACKNOWLEDGMENTS

This paper was condensed from a report presented by the junior authors in partial fulfilment of the requirements for the degree of Master of Science in Sanitary Engineering from the University of North Carolina.

APPENDIX I.—IDENTIFICATION OF RESERVOIR NUMBER GIVEN ON DATA SHEETS

North Carolina:

6-1	Lake Apex, Swift Creek, Apex.
6-2	Franklinton Reservoir, Sallie-Keaney Creek, Franklinton.
6-3	Lake Brandt, Reedy Fork, Greensboro.
6-4	Highpoint Reservoir, Deep River, High Point.
6-5	Lake Michie, Flat River, Durham.
6-6	Sanford City Reservoir, Lick Creek, Sanford.
6-7	University Lake, Morgan Creek, Chapel Hill.
6-8	Roxboro City Lake, Satterfield Creek, Roxboro.
6-9	Burlington Municipal Reservoir, Stoney Creek, Burlington.
6-10	Walnut Cove Reservoir, Dan River, Walnut Cove.
7-5	Albemarle City Lake, Long Creek, Albemarle.
7-6	Cannon Lake, Buffalo Creek, Kannapolis.
7-7	Lake Concord, Chambers and Rose Branch, Kannapolis.
7-8	Entwhistle No. 3, Hitchcock Creek, Roberdell.
7-9	Eury Reservoir, Little River, Troy.
7-10	High Rock Reservoir, Yadkin River, Salisbury.
7-11	Lake Lee, Richardson Creek, Monroe.
7-12	Pee Dee Manufacturing Co., Hitchcock Creek, Rockingham.
7-13	Salem Reservoir, Salem Creek, Winston Salem.
7-14	Norwood Lake, Pee Dee River, Mt. Gilead.
7-15	Lexington Reservoir, Leonard's Creek, Lexington.

20-11	Fontana Reservoir, Little Tennessee River, Fontana.
20-12	Cheoah Reservoir, Little Tennessee River, Tapoco.
20-16	Chatuge Reservoir, Hiwassee River, Hayesville.
20-18	Hiwassee Reservoir, Hiwassee River, Murphy.
South Carolina:	
7-1	Chester Reservoir, Sandy River, Chester.
7-2	Lancaster Reservoir, Turkey Quarter Creek, Lancaster.
7-3	Spartanburg Municipal Reservoir, S. Pacolet River, Spartanburg.
7-4	Appalachie Reservoir, South Tyger River, Greer.
8-1	Lake Isaqueena, Six Mile Creek, Clemson.
Georgia:	
20-17	Nottely Reservoir, Nottely River, Blairsville.
8-2	Lloyd Shoals, Ocmulgee River, Jackson.
11-1	Newman Reservoir, Bolton Mill Creek, Newman.
12-1	Sequoyah Reservoir, Small Brothers, Jasper.
12-2	White Manganese No. 6, Pettit Creek, Cartersville.
Alabama:	
12-3	Lake Auburn, Tributary of Town Creek, Auburn.
12-4	Lay Reservoir, Coosa River, Clanton.
12-5	Lake Purdy, Little Cahaba River, Birmingham.
13-1	Bayview Reservoir, Village Creek, Birmingham.
13-2	Lake Harris, Yellow Creek, Tuscaloosa.
18-4	Guntersville Reservoir, Tennessee River, Guntersville.
18-5	Wheeler Reservoir, Tennessee River, Town Creek.
18-6	Wilson Reservoir, Tennessee River, Florence.
Mississippi:	
15-26	Arkabutla Reservoir, Coldwater River, Arkabutla.
Tennessee:	
18-1	Radnor Lake, Other Creek, Nashville.
18-7	Pickwick Landing, Tennessee River, Pickwick.
20-5	Cherokee Reservoir, Holston River, Jefferson City.
20-6	Nolichucky Reservoir, Nolichucky River, Greenville.
20-7	Douglas Reservoir, French Broad River, Sevierville.
20-13	Caryville Reservoir, Cove Creek, Caryville.
20-14	Norris Reservoir, Clinch River, Norris.
20-19	Apalachia Reservoir, Hiwassee River, Farner.

20-21 Ocoee No. 3, Ocoee River, Ducktown.

20-22 Ocoee No. 1, Ocoee River, Parksville.

20-23 Chickamauga Reservoir, Tennessee River, Chattanooga.

20-24 Hales Bar Reservoir, Tennessee River, Jasper.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3134

REGLA STEAM ELECTRIC STATION

By George T. Ingalls,[1] M. ASCE

SYNOPSIS

This paper describes the design of a four unit steam electric station. Unusual features include design of foundations to cope with stiff clay, rock strata and slag deposits, outdoor-type construction, strengthening gantry crane and ventilating louvres for turbo-generator housings.

INTRODUCTION

The Regla Steam Electric Station is located in the town of Regla, on the eastern shore of Havana Harbor, in Cuba. The site had been used as a recreation field (see Fig. 1). The lot is approximately 430 ft by 440 ft or about 4 1/2 acres. It is bounded by the waters of Havana Harbor on the west and north side.

The town of Regla was a haven for pirates in colonial days and any sailing ships that docked there were subject to raids. It was quite common for these ships to be stripped of their copper bottoms.

The site was previously occupied by a large iron foundry. There were a number of slag deposits buried in the ground and, also, around the shore line. Just off the west shore was a submerged marine railway.

A number of sites were investigated before Regla was finally selected. No site has all the ideal conditions such as water supply, location near large power users, good sub soil for foundations, general system requirements, land values,

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Note.—Published essentially as printed here, in June, 1960, in the Journal of the Power Division, as Proceedings Paper 2520. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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transportation, and a good supply of labor. The Regla site, however, was ideal as to situation with adequate cooling water, good supply of labor, and good transportation (a ferry line is adjacent to the plant within 10 minutes commuting distance to Havana). The main drawback was its restriction as to size of lot. It required considerable juggling (See Fig. 2) to get four units with an ultimate capacity of 209,000 kw and all auxiliaries on 4 1/2 acres. This includes substation, oil tanks, office building, machine shop, chemical storage, and laboratory building. In addition to this, the Cuban government required a 20 ft clear right of way on the harbor side for emergency vehicles. This is called the "Zone of Vigilance."

The location has the advantage of being accessible for barges to unload fuel oil directly into the two 15,000-barrel storage tanks. In 1958, it was suggested that high viscosity oil be used and pumped from storage tanks about two miles away. This oil is the residue of the refinery and requires a high temperature for liquid flow.

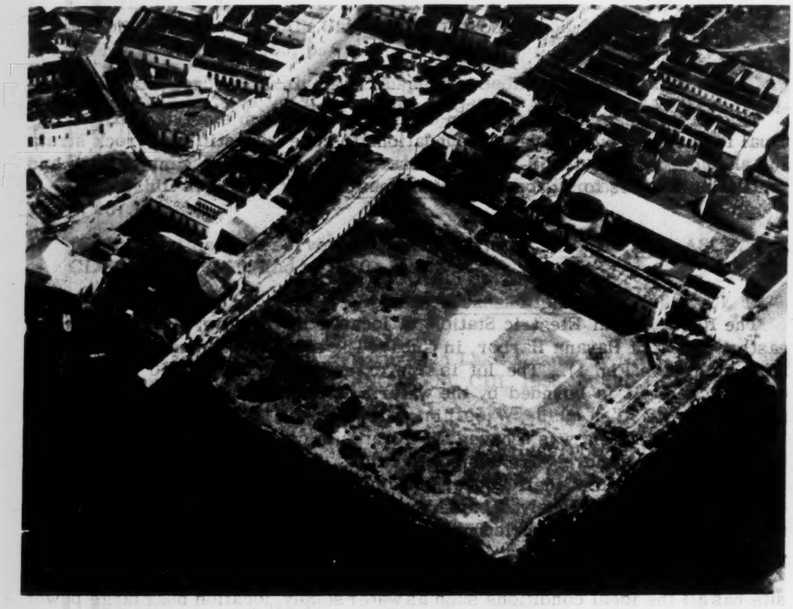

FIG. 1.—PLANT SITE BEFORE CONSTRUCTION

DESCRIPTION OF PLANT

The Regla Steam Electric Station is of the "semi-outdoor" type, composed of steel frame and concrete floors. The turbine and auxiliary bays have no side walls. The turbo-generator is completely housed on the operating floor and a 50-ton gantry crane straddles this housing to service the turbo-generator through removable hatch covers on the roof. The oil-burning boilers are an outdoor installation (two boilers for each unit) along with their fans and auxiliary equipment. The remaining structures consist of a combined office and electrical building, machine shop, laboratory, and chemical storage building. All of these buildings are of reinforced concrete with flush brick spandrel walls, cement coated.

Comparative costs were made of a closed station (walls, roof and overhead crane) and a semi-outdoor station (no walls or roof, except housing for turbo-generator and gantry crane). There was an estimated savings in excess of 1/2 million dollars using the semi-outdoor station. Total cost of Units 1 and 2, including substation, are $6,600,000 and $6,730,000, respectively.

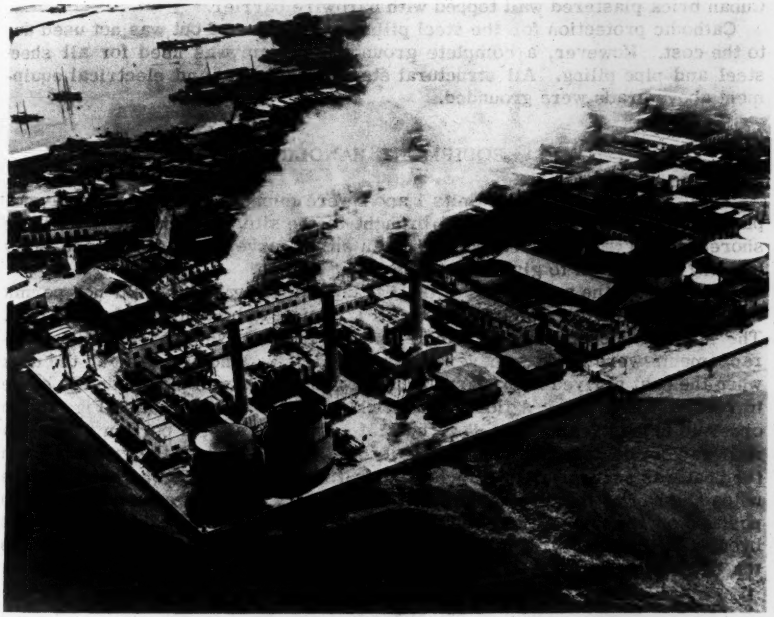

FIG. 2.—FINISHED PLANT—UNITS 1, 2 AND 3 OPERATING

Service water is supplied from the municipal water supply to a 640,000 gal reinforced concrete storage tank, placed underground. The storage tank was necessary due to the uncertainty of a continuous supply of city water. Water is demineralized before being used as boiler make-up. The storage tank is compartmented for separate storage of potable water. Circulating water screen chamber and intake canal is located on the north side of the plant while the discharge canal is on the west side.

To retain the fill on the north and west boundaries of the site, a sea wall of interlocking steel sheet piling was driven to a depth of 40 ft to 50 ft with tie backs 15 ft on centers, consisting of four 1 1/4 sq in. tie rods encased in a 12 sq in. concrete envelope. These were anchored back to a concrete dead man supported by two steel pipe batter piles. The sheet piles were capped with concrete which extended 2 ft below mean low water. A retaining wall constructed on top of the cap retains yard fill. The pile cap serves as a waler and as protection for piles above mean low water.

The switchyard and transformers are adjacent to the electrical building on the east side. All major equipment of the switchyard, including the steel structure, is supported by piles. Transformers rest on rails which extend to the unloading bay.

The highest variation of tide on record in Havana Harbor is about 2 ft or el 102. Finished yard grade of el 107 ft, gives a margin of 5 ft.

The plant area on the street sides (south and east side) is enclosed with Cuban brick plastered wall topped with barbwire barrier.

Cathodic protection for the steel piling was discussed but was not used due to the cost. However, a complete grounding system was used for all sheet steel and pipe piling. All structural steel, mechanical and electrical equipment above grade were grounded.

EQUIPMENT HANDLING

The turbo-generators for Units 1 and 2 were unloaded at the dock in Havana, placed on railroad flat cars, and brought to the site. The bridges had to be shored, to carry the increased load. At a sharp curve a corner of a one-story building was cut off to give the necessary clearance.

For Unit 3, the authorities at the Havana unloading dock would not permit the generator stator which weighed 118 1/2 tons, to be unloaded from a boat. The nearest dock that could handle this load was at New Orleans, La. Arrangements were made to have a converted L.S.T. landing barge at the dock when the boat arrived from Germany. A low-boy truck was placed on the barge to receive the stator. A floating derrick made the transfer, (See Fig. 3). In order to unload from the L.S.T. boat at the Regla site, a portion of the sea wall was cut down to allow the ramp of the L.S.T. to seat on the ground. When the boat was securely tied to anchor posts, the low-boy truck was hauled to the unloading bay of the turbine building. The gantry crane lifted and placed it in position on the turbine pedestal. The remainder of the turbo-generator was brought to Havana by the overseas boat, unloaded at the dock, and trucked to the Regla site.

INTAKE STRUCTURE

The intake structure consists of screen chamber, divided into four sections. A single 4-unit intake canal is open for the major portion and is slabbed over at the circulating water pumps. The entire structure is of reinforced concrete construction supported on wood and steel piles. The screen chamber has a concrete skimmer wall at the entrance extending 3 ft below mean low water, stop log guides for the reinforced concrete stop logs. Following these are coarse and revolving screens. When the revolving screens are removed for servicing a set of fine screens are placed in the fine screen guides.

The canal was designed as a rigid frame structure with concrete walers and struts near the top.

A 20-ton gantry crane services the revolving screens. The crane rail is carried on a concrete girder runway extending 54 ft on the easterly side of the screen chamber. When construction started on the intake chamber, excavated material was used to form an earth dyke. Due to flood tide conditions during construction, the dyke collapsed at the harbor entrance and flooded the work area. Sheet piling was then driven at the entrance and after construction was completed, it was cut down to the invert of the chamber. The invert of the screen chamber is el 88 ft, and the invert of the canal is el 90 ft. The top of

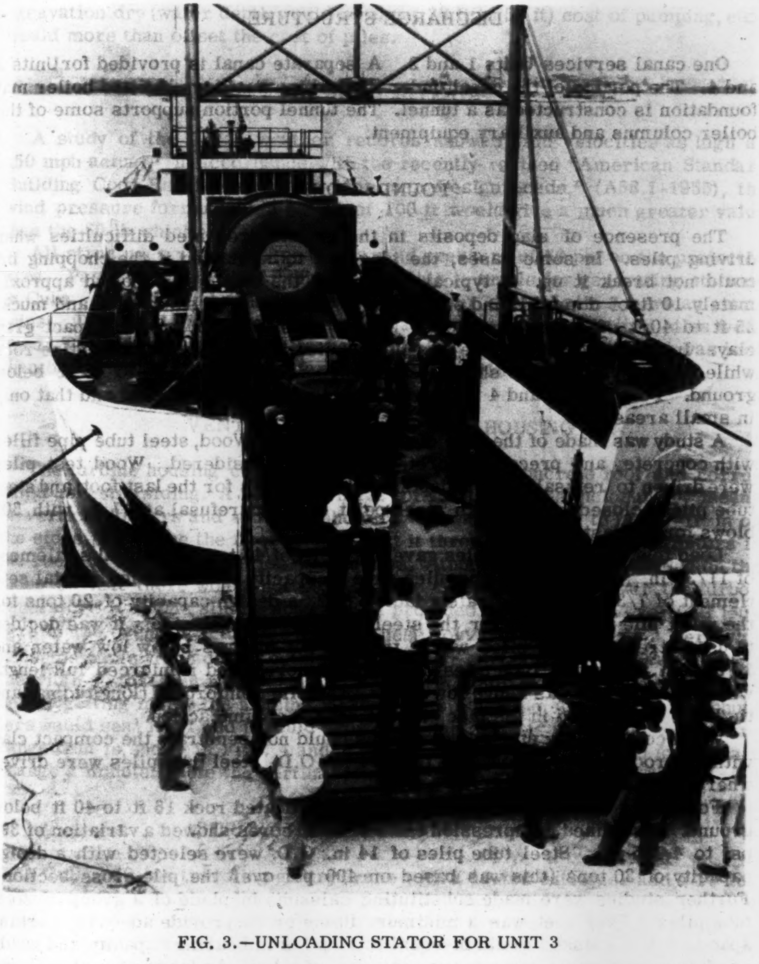

FIG. 3.—UNLOADING STATOR FOR UNIT 3

the deck slab is at el 107 ft. The intake canal decreases in width from 20 ft at Unit 1, to 8 ft at Unit 4.

Due to the close proximity of the foundation for the electrical building to the intake canal, it was necessary to build the intake canal before driving piles for the electrical building.

DISCHARGE STRUCTURE

One canal services Units 1 and 2. A separate canal is provided for Units 3 and 4. The portion of the canal under the turbine auxiliary bay and boiler mat foundation is constructed as a tunnel. The tunnel portion supports some of the boiler columns and auxiliary equipment.

FOUNDATIONS

The presence of slag deposits in the ground presented difficulties when driving piles. In some cases, the slag had to be blasted if the chopping bit could not break it up. A typical boring for the Unit 1 site showed approximately 10 ft of dump fill and silt, 10 ft to 25 ft of gray sandy clay and muck, 25 ft to 40 ft of yellow sandy clay, and 40 ft to 100 ft of hard compact gray clay. In the Unit 1 area there was little indication of gray or serpentine rock while borings for Unit 2 showed solid serpentine rock 16 ft to 40 ft below ground. For Units 3 and 4 rock was 30 ft to 60 ft below ground and that only in small areas.

A study was made of the type of pile to be used. Wood, steel tube pipe filled with concrete, and precast concrete piles were considered. Wood test piles were driven to refusal at 24-ft depth, with 300 blows for the last foot; and steel tube piles, closed-ended, with steel point reached refusal at 27 ft, with 500 blows for the last foot.

Load tests for the wood piles gave a capacity of 40 tons or a total settlement of 11/32 in. and the steel tube piles gave a capacity of 80 tons for a total settlement of 7/32 in. With a safety factor of 2, a design capacity of 20 tons for the wood pile and 40 tons for the steel tube pile was selected. It was decided to use untreated wood piles where the pile cut-off was below low water and 10 3/4 in. O.D. steel tube piles filled with concrete and reinforced full length where pile cut-off was above low water. The pile reinforcing (longitudinal and tie bars) was placed in the pipe piles to secure column action.

When construction started wood piles could not penetrate the compact clay without brooming. As a result, ten-3/4 in. O.D. steel tube piles were driven where wood piles were specified.

For Unit 2 installation, the soil borings indicated rock 16 ft to 40 ft below ground. Unconfined compression tests of rock cores showed a variation of 398 psi to 4,295 psi. Steel tube piles of 14 in. O.D. were selected with a design capacity of 30 tons (this was based on 400 psi over the pile cross section). Further studies were made substituting caissons in place of a group of steel tube piles. Four feet was a minimum diameter to provide adequate working space. A 4-ft caisson would be equal to 11 piles in bearing capacity and would provide considerable savings over the cost of piles. An inspection of the profile of the serpentine rock showed considerable slope in a number of cases. In view of this, it was specified that caissons should be seated in the rock to get a level bearing. If the bearing value of the rock was less than 400 psi the caisson was to be belled out at the bottom. Where loads were spread out or much less than the capacity of the minimum size caisson, steel tube piles filled with concrete and reinforced full length were used. All steel pipe piles were closed-ended with cast steel points.

It was not economical to use caissons for Units 3 and 4, as the difficulties encountered, such as excavating in a confined space at greater depth, keeping excavation dry (water depth would be about 30 ft to 50 ft) cost of pumping, etc., would more than offset the cost of piles.

DESIGN CRITERIA

A study of the Cuban weather records showed wind velocities as high as 150 mph actual. In accordance with the recently revised "American Standard Building Code Requirements for Minimum Design Loads," (A58.1-1955), the wind pressure formula at a height of 100 ft would give a much greater value than the 50 lb which was used.

All structures were designed for a wind pressure of 50 psf on the projected area. Past experience has indicated that this is ample for generating stations in Cuba. Lesser wind pressures are used in our other Latin American countries. To provide for this lateral force at the foundations, batter piles were used. This applied to all structures where the ratio of height to width was less than one, or where the lateral force was greater than 1,000 lb per pile.

VENTILATION OF TURBINE HOUSING

The turbine housing consisted of a steel frame, concrete roof and corrugated asbestos siding. A study was made for ventilating the enclosure by means of vertical louvers and movable shutters. It was desirable to bring air in on the sides just above the floor and exhaust it through eight power ventilators in the roof for each unit. Stationary windows, without vents, were provided on the sides so that there would not be any short circuiting of air over the turbogenerator. A unique sliding shutter was provided to close off the vertical louvers in the event of a driving rain. These louvers consisted of a series of asbestos cable trays on the outside and another series on the inside offset half the width so that any rain laden air coming in would have to make two passes before getting inside the housing. If there was much moisture in the air, shutters would seal it off and it would drop down and drain to the outside. This arrangement is shown in Figs. 4 and 5. The ventilation was designed for one air change a minute. Note the vertical louvers under the windows in Fig. 5.

STRENGTHENING GANTRY CRANE

A Colby, 50-ton traveling gantry crane with a 10-ton outrigger and a 10-ton auxiliary hook was placed on the operating floor, which is the same as the roof level. The 10-ton outrigger services the circulating water pumps and the heaters. The generator stator for Unit 1 weighed 58 tons. The Unit 2 generator stator weighed 92 tons. To handle this overload, it was necessary to pick up the load from the main drum through a needle beam and distribute it through sheaves and cable to each end of the crane bridge. This brought half of the

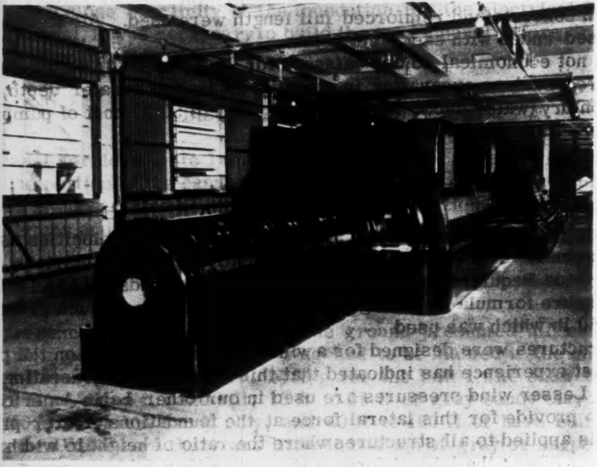

FIG. 4.—INTERIOR VIEW OF TURBINE HOUSING

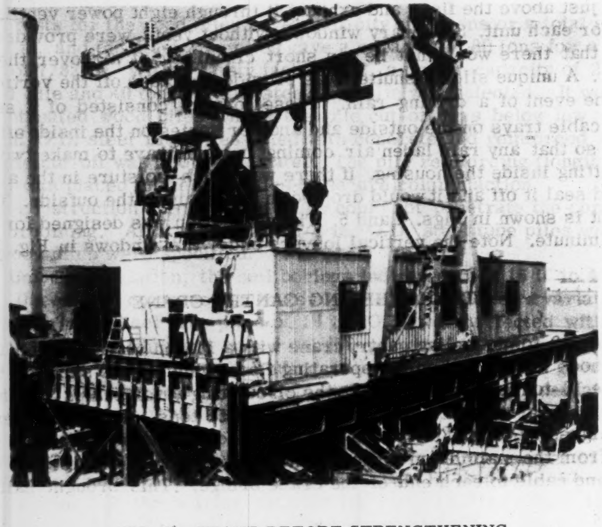

FIG. 5.—CRANE BEFORE STRENGTHENING

load directly into each gantry leg. Heavier cable replaced the existing and additional stiffeners, and bracing was installed (see Fig. 6). The maximum wheel load of a crane is determined by placing the lifted load at one end of the crane bridge. With the arrangement that has been outlined, the increased original wheel load was 18%. A check was made of the stresses in the steel framing and foundations for the weight of stator for Units 3 and 4 and they were found within the safe limits.

SUBSTATION

It has been general practice in substations to place the transformers on rails so that the transformer can be readily moved when repairs are needed.

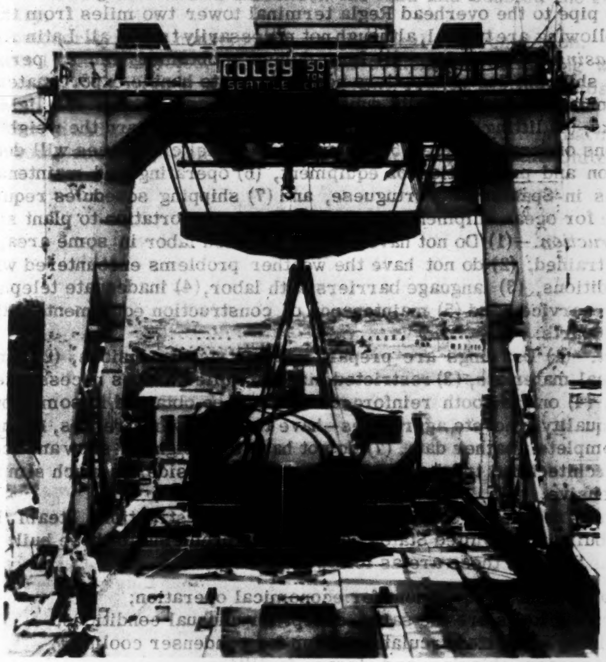

FIG. 6.—50 TON CRANE

Where the substation is adjacent to the generator plant, the crane runway is extended one bay beyond the main building for servicing transformers and other equipment. At Regla this extension is adjacent to the machine shop. When a substation is away from the generating plant a transformer tower is erected on the site and the transformer is rolled underneath and serviced by an electric hoist. All oil filled transformers are separated by reinforced concrete fire walls. Crushed stone was placed around the transformers for oil drainage in the event of fire. The depth of this crushed stone was based on releasing the total quantity of oil through 50% voids in the stone.

A study was made for reducing the cost of formwork and standardizing the size of transformer fire walls. Types considered were (a) reinforced concrete piers with precast concrete panels, (b) reinforced concrete piers with concrete block filled with concrete and reinforced, (c) battered, reinforced concrete wall with a varying base dimension and 8 in. at the top, and (d) precast concrete—post tensioned.

It was found that type (c) was the most practicable and economical, if the size of transformer walls were standardized. By using the manufactured steel frame plywood forms in 2-ft widths, with varying lengths, clamps and ties, there was a saving in the cost of formwork, depending on the number of reuses. A standard firewall drawing, showing wall thinkness and reinforcing, was made for all sizes that are in common use. The sizes ranged from 14 ft to 28 ft in height, and for a unit length of one ft.

The distribution of power from the switchyard was underground through oil pressure pipe to the overhead Regla terminal tower two miles from the plant.

The following are typical, although not necessarily true of all Latin America.

Purchasing and Shipping.—(1) Long delays in obtaining import permits, (2) delays in shipping due to suppliers not equipped to handle export material, (3) packing must be exceptionally substantial where shipments are lightered to port docks, (4) lifting facilities at port of entry will govern the weight and/or dimensions of largest piece, (5) railroad and tunnel clearances will determine fabrication and preparation of equipment, (6) operating and maintenance instructions in Spanish or Portuguese, and (7) shipping schedules require long lead time for ocean shipment, local customs, transportation to plant site.

Construction.—(1) Do not have qualified skilled labor in some areas—labor must be trained, (2) do not have the weather problems encountered with subzero conditions, (3) language barriers with labor, (4) inadequate telephone and telegraph service, and (5) maintenance of construction equipment—difficult to get spare parts.

Design.—(1) Drawings are prepared in metric dimensions, (2) limited by use of local materials, (3) restricted sizes of rolled shapes necessitate welded sections, (4) only smooth reinforced bars can be obtained in some countries, (5) poor quality concrete aggregates—have to use lower stresses, (6) unable to obtain complete weather data, (7) do not have to design for snow and ice loads, and (8) architectural treatment, both inside and outside, is much simpler and less expensive.

In some respects, concerning design and construction, Steam Electric Stations built in the United States have similar problems to those built in Latin America. Some of these are as follows:

1. Arrangement of equipment for economical operation;
2. operating loads are the same, except for unusual conditions;
3. obtaining adequate circulating water for condenser cooling;
4. fuel sources, shipping, and storage facilities;
5. location of plant on ground above maximum probable floods;
6. location of plant on the best available soil conditions;

7. location of plant for service by railroad, highway or river navigation;
8. location of plant and switchyard for outgoing transmission lines;
9. adequate ventilation of plant; and
10. sufficient room for construction and for future expansion.

CONCLUSIONS

When the Regla Steam Electric Station was planned, the demand for power was far greater than the supply. One of the oldest stations, the Consolidated Steam Electric Station which is across the harbor, (in Havana), needed major overhauling. It was planned to do this work when Regla Unit 1 was completed. The demand, however, was even greater than expected and so Unit 2 was started and following this was Unit 3. During the last 10 years, in the Havana area, a large number of apartments, office buildings, residences, and some hotels have been built. Greater use of air conditioning, electrical appliances, and industrial expansion have increased the load and boosted the economy of the country.

The outdoor-type of construction seems ideal for ventilation. However, when Unit 1 was completed, the operators complained of getting wet during driving rains. Two sides were enclosed with corrugated asbestos on steel frame with ventilated steel sash.

The design and construction of steam electric stations built in Latin America present some unusual problems that are not encountered in the United States.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3135

FURNAS HYDROELECTRIC PROJECT

By James W. Libby,[1] M. ASCE

SYNOPSIS

This paper gives background data and describes the general design arrangement of Furnas Hydroelectric Project which is now under construction on the Rio Grande in the state of Minas Gerais, Brazil. Ultimately, Furnas will have an installed capacity of 1,200,000 kw, placing it among the world's largest hydroelectric developments.

INTRODUCTION

Brazil's population and industry is concentrated along 4,600 miles of Atlantic coastline. The states of Sao Paulo, Minas Gerais, Rio de Janeiro, and Espirito Santo have had, as late as 1954, about 45% (25 million) of the nation's population. Also located within these states was about 80% (only 2-1/4 million kw) of the entire installed electric generating capacity. The demand for additional power is urgent and increasing at a rate in excess of 10% compounded annually.

Thermal plant construction would appear to be an immediate method of overtaking the load demand, and private utilities have utilized such construction to a limited extent. However, as fuel must be imported, it substantially increases the cost of this type of energy. Nuclear energy with its present high cost is precluded as a solution to the immediate demand.

Brazilian officials have studied a number of major hydroelectric sites and have concluded that the early development of some of these sites presents the

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Note.—Published essentially as printed here, in April, 1960, in the Journal of Power Division, as Proceedings Paper 2430. Positions and titles given are those in effect when the paper or discussions was approved for publication in Transactions.

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best answer to their power needs. One of the most desirable, located along the Furnas reach of the Rio Grande, is presently under construction.

Preliminary investigations of the Furnas site were initiated in 1954 at the request of Centrais Eletricas de Minas Gerais (CEMIG). When detailed field investigations began later in 1956, CEMIG invited Brazilian Traction's Sao Paulo Light and its technical unit (COBAST) to assist. As the power situation grew more critical, the Brazilian Federal Government formed a new organization, on September, 1957, known as Central Eletrica de Furnas (FURNAS). It is comprised of the Federal Government as a major sponsor, the states of Sao Paulo (DAEE) and Minas Gerais (CEMIG), Sao Paulo Light and Cia. Paulista de Forca e Luz. FURNAS has its engineering headquarters in Rio de Janeiro with a field office at the Furnas site.

REGIONAL POWER DEVELOPMENT

The market area and principal load centers are shown on Fig. 1. Recent estimates made by major utilities serving this area indicate the power requirements will be about 4,000,000 kw in 1960, 6,400,000 kw in 1965, and in excess of 10,000,000 kw by 1970.

This area had an installed capacity of 2,225,000 kw in 1954, and 2,500,000 kw at the end of 1957.

Major generating additions, other than the Furnas project, contemplated within the next several years are limited. The Peixoto plant downstream from Furnas is adding 90,000 kw. The Tres Marias project now under construction will have two units (130,000 kw) in operation by 1961 with two additional units to be added in the near future. The final installation will consist of eight units totalling 520,000 kw. One thermal installation is currently undergoing an expansion of 250,000 kw.

The region is faced with two other problems in its expansion program. One problem arises because service is supplied at both 60 cycles and 50 cycles, hampering interconnections. The other problem is the absence of a major transmission grid interconnecting the various power producing and distributing facilities. Little advantage can be taken of load diversity, surplus capacity, and energy between areas. Major additions to the transmission grid are planned coincident with the completion of the Furnas and Tres Marias projects.

SITE LOCATION

The Furnas site is located in the southwestern part of the State of Minas Gerais on the Rio Grande, a major tributary of the Parana (Fig. 1). It is about 320 km (200 miles) north of Sao Paulo, 280 km (175 miles) southwest of Belo Horizonte, and 400 km (250 miles) northwest of Rio de Janeiro.

Upstream from Furnas are two existing small developments with potential sites for two more on the main stem of the Rio Grande and another on a tributary. The aggregate installed capacity and usable storage of these are small in comparison to the Furnas project and will have an insignificant effect on Furnas operation.

Immediately downstream from the Furnas site is the existing Peixoto plant of Empresas Eletricas. After the completion of the Furnas project, the Peixoto plant will receive material benefits due to increased river regulation. Next downstream from Peixoto is the Estreito site which is under investigation by

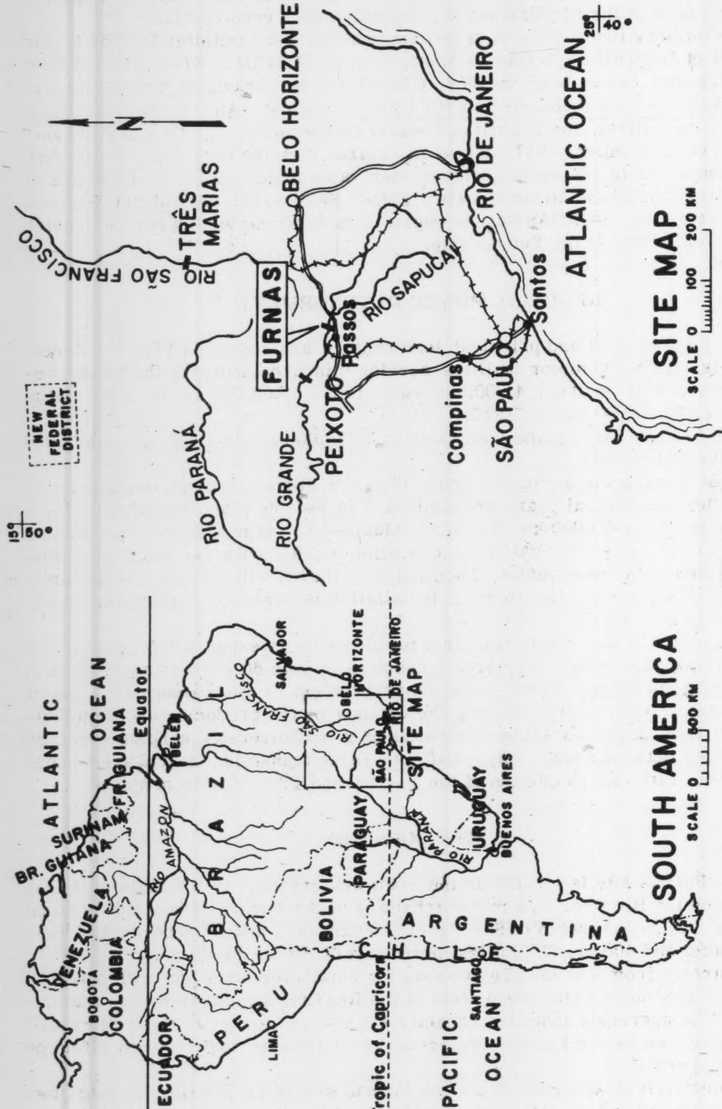

ATLANTIC OCEAN
Equator
BR. GUIANA
SURINAM
FR. GUIANA
VENEZUELA
BOGOTA
COLOMBIA
ECUADOR
PERU
LIMA
BRAZIL
RIO AMAZON
BELEM
SALVADOR
RIO SÃO FRANCISCO
BELO HORIZONTE
RIO DE JANEIRO
SÃO PAULO
SITE MAP
BOLIVIA
PARAGUAY
Tropic of Capricorn
ARGENTINA
CHILE
SANTIAGO
PACIFIC OCEAN
URUGUAY
BUENOS AIRES
RIO PARANÁ
SOUTH AMERICA
SCALE 0 500 KM
15° 50°
NEW FEDERAL DISTRICT
N
RIO SÃO FRANCISCO
TRÊS MARIAS
BELO HORIZONTE
FURNAS
RIO PARANÁ
RIO GRANDE
PEIXOTO
Passos
RIO SAPUCAÍ
Compinas
SÃO PAULO
Santos
RIO DE JANEIRO
ATLANTIC OCEAN
25° 40°
SITE MAP
SCALE 0 100 200 KM

FIG. 1

the Empresas Eletricas. This site when developed will probably have an installed capacity of about 800,000 kw. It will depend upon Furnas and Peixoto for regulation of stream flow. There are perhaps ten other potential sites downstream from the Estreito site which will benefit from regulation by the Furnas reservoir.

The closest railroad terminates about 32 km (20 miles) downstream from the Furnas site at the town of Passos. An existing state highway runs from Passos and passes within 8 km (5 miles) of the site. Improvement of both railroad and highway were required as well as the construction of an access road.

INVESTIGATIONS

Based on information obtained from early field work, preliminary office studies were performed to compare a number of alternate layouts at various sites. A single high dam at the lower end of the Furnas Canyon proved to be the most attractive. A report was released in September, 1955, which included recommendations for an expanded investigation of the site and reservoir area. The recommendations included surface and subsurface investigations, a drilling program, topographic surveying of dam and reservoir area, search for suitable construction materials and studies of alternate means of transport and access. This work was started almost immediately.

Meanwhile, the geologic and hydrologic data were reviewed, reservoir area-capacity curves redrawn from improved field data, and new power studies were made. During this period a number of consultants visited the site. Recognized authorities in geology, soil mechanics, heavy construction, hydroelectric power development, etc., were engaged.

SELECTION OF TYPE OF DAM

Considerable study was devoted to the selection of the best type of dam. It was determined early in the program that the differential cost between various types of structures would be relatively minor. Final selection of the type of dam, therefore, would be largely governed by subsurface findings and availability of construction materials.

The site itself was susceptible of development by either a fill-type dam or one of several concrete structures. Early indications favored a concrete gravity dam. Later, exploratory drilling revealed that the right abutment would require excessive excavation to prepare an adequate foundation for a concrete structure. Furthermore, much of the aggregate would have to be manufactured. Brazil has limited cement-producing facilities, and these would be severely taxed to produce the large quantities of cement required for a concrete structure and still meet other commitments. As more materials information was accumulated, it became apparent that a fill-type structure had distinct advantages. Also, the construction scheme would be simplified and would permit a faster construction schedule than would be possible with a concrete structure. The decision was made to design the project substantially as shown in Fig. 2.

HYDROLOGY AND METEOROLOGY

The drainage area of the Rio Grande upstream from the Furnas site is about 51,000 sq km (20,000 sq miles). The river rises in the Serra de Mantiqueira in South Central Minas Gerais and flows generally westward to its confluence with the Parana.

A reliable gage has been maintained at Sao Jose da Barra, a short distance above the dam site, since 1930. By correlation with other stations, records were extended to mid-1928 in order to include 1929, which was a year with extremely high rainfall. Inflow between Sao Jose da Barra and the dam site is negligible, so the gage records were used for natural discharges at Furnas. Mean monthly flows have varied from a minimum of about 210 cms (7400 cfs) to a maximum of 4130 cms (146,000 cfs). The mean flow for the period of record is approximately 900 cms (31,700 cfs).

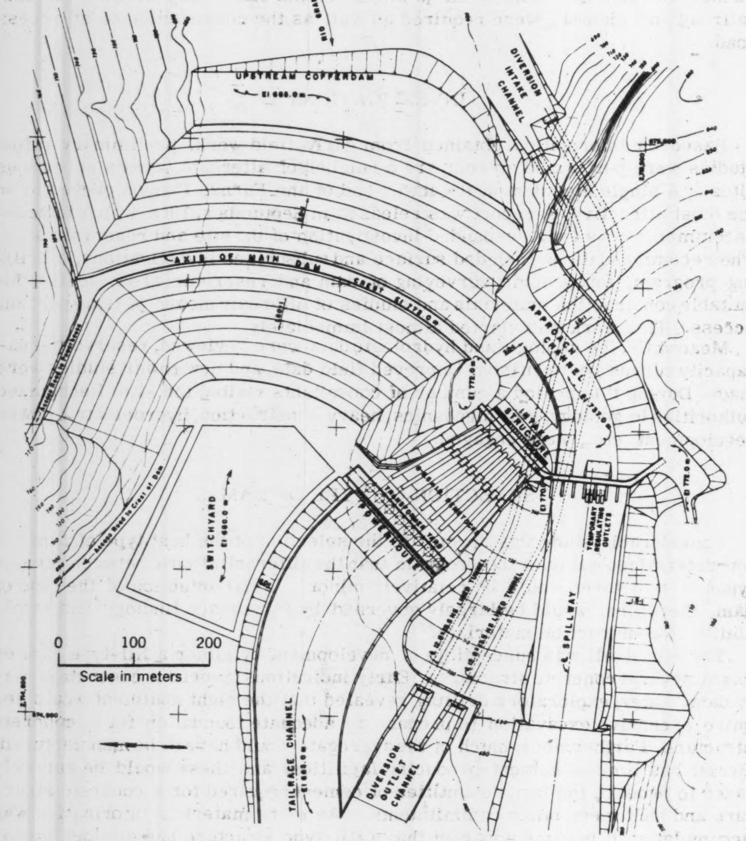

RIO GRANDE
UPSTREAM COFFERDAM
El 688.0m
DIVERSION INTAKE CHANNEL
AXIS OF MAIN DAM
CREST El 772.0m
APPROACH CHANNEL El 737.0m
El 772.0m
INTAKE STRUCTURE
POWERHOUSE
TRANSFORMER YARD
SWITCHYARD El 680.0m
TEMPORARY REGULATING OUTLETS
SPILLWAY
El 772.0m
TAILRACE CHANNEL El 665.0m
DIVERSION OUTLET CHANNEL
Access Road to Passos
Access Road to Crest of Dam
0 100 200
Scale in meters

FIG. 2.—GENERAL PLAN OF FURNAS PROJECT

The flow record includes one extremely high year, and two protracted low periods. The first low period began about mid-1932 and extended into 1935. The second began in mid-1952 and lasted until early 1957. For purposes of power production, the latter was more critical because of its longer duration.

Flood potential is determined by the flow of tropical maritime air. The basin is mountainous, with maximum elevations reaching nearly 3,000 m (10,000 ft) and the headwaters are only 105 km (65 miles) from the Atlantic ocean. Circulation of air over Brazil is governed largely by a permanent high pressure area over the Atlantic with its center east of Uruquay, and by the position of the intertropical front. When it swings south, as it usually does in the January-March period, this front lies in a V-shaped pattern over the continent, with its southern tip extending to the northern boundary of Paraguay. Tropical maritime air from the northeast flows up the Amazon Valley toward the front and causes intense precipitation there. Simultaneously, maritime air from the east, also flowing toward the front, strikes most of the Brazilian coast. Air from the northern flow can enter the valley of the Sao Francisco when the intertropical front lies unusually far to the south. Ordinarily very little of this air will reach the Rio Grande basin. Air from the eastern flow will cross the Rio Grande whenever behavior of the front is normal, and the flow will become stronger as the front approaches the basin.

The basin is protected on the north and east from low level maritime air by 5,000-ft barriers. Except near the headwaters of the Sao Francisco and Rio Grande, the northern barrier is quite broad, while that on the east consists of a single ridge near the coast. Convectively unstable air from the east, carried by strong winds aloft, can cause heavy precipitation over the whole Rio Grande basin. However, the rain potential of these storms is lower than those from the north.

Although locally there are areas to the south that experience intense rainfall, general topographic configuration combined with the necessary easterly wind precludes the possibility of extended storms from the west or southwest. It is of interest to note that a careful study of the recent severe floods in Uruguay is currently being conducted with reference to storms approaching the Rio Grande basin from the south.

It was concluded that a spillway design flood would have to be produced by a storm from the north, with some lesser effect from a simultaneous easterly storm. The computations showed that such a storm would result in a 10,000-yr flood with a peak daily inflow of 18,000 cms (635,000 cfs), and a ninety-day volume of 945,000 cms-days (33,400,000 cfs-days).

GEOLOGY

Rock formations in the Furnas region belong to the Minas and Itacolomi series which were derived from sediments deposited in a Pre-Cambrian inland sea known as the Minas Sea. These sediments were subjected to considerable folding and shearing. Fine beach sands became sandstones, which later were fused by high temperatures. Finer sediments became indurated into shales, which later were metamorphosed into the sericite schists that are found interbedded with quartzite. Quartzites rich in mica represent sands that were mixed with silt or clay.

The Minas series is composed chiefly of sericite or chlorite phyllites, medium-grained, impure quartzites, magnesian limestone and itabirites. The

Itacolomi series includes great thicknesses of thin-bedded, fine-grained white, micaceous quartzites. They grade into the fine-grained laminated rocks rich in schistose elements that are known as Itacolomites.

First visits to the site were made in mid-1954. An exploratory drilling program was initiated in October, 1955, and was largely completed by mid-1958. An extensive grid of drill holes was logged, and geologic profiles and sections were prepared.

The left abutment at the Furnas site is composed of a finely grained highly jointed quartzite. The surface of suitable bedrock averages from 5 to 10 m in depth below natural ground surface over most of the left abutment except at the spillway area where maximum depth is about 35 m (115 ft). The general strike is North 25° West, with the dip 15° to 20° to the Southwest, or downstream and away from the river channel. Bedding planes form joints consisting of very thin layers of clay or fine silts which have been metamorphosed into micaceous planes.

In the main river channel there is a deep diagonal fault-formed trough. This has been intruded by a diabase dike that has gouge and fractured rock on both sides.

The right abutment is composed of thick beds of thinly laminated quartzites separated by relatively deep bands of micaceous schists that are fractured and friable near the surface. There has been considerable decomposition and weathering of the schists and a talus slope has formed at this abutment. Removal of the talus and weathered material will be necessary.

Areas containing decomposed schist are present at both abutments. It is expected these materials will be impervious, except where extreme weathering has taken place. Joints in the quartzites usually are tight and little grouting will be required in most areas. Water testing indicates little difficulty will be experienced with leakage.

MATERIALS INVESTIGATIONS

As investigations progressed it became apparent that a fill-type structure was better suited to site conditions than was a concrete structure. This resulted in an intensive search to locate sufficient quantities of the various types of material to be used in the dam. At the same time a testing program was instigated in which routine classification and strength tests were carried out to determine the characteristics of all such materials. Specialists in engineering geology and soil mechanics were consulted. Field tests have shown that most excavated material will be suitable for the random fill, and that rock excavation in the intake, spillway and powerhouse areas can be utilized for the rock shell portion of the dam.

Because of the slabby nature of the quartzite fragments to be used for the rock shells, it was considered advisable to make special investigations of the strength characteristics of this material. The U. S. Bureau of Reclamation in Denver was retained to perform large scale triaxial compression tests on the rockfill using high all-around pressures. The assumed strength parameters used in the preliminary stability studies would be reviewed and modified where necessary after the results of these tests became available.

Suitable earth core material is available on the right bank about a mile upstream from the dam axis and on the left bank about 4 miles from the site. Adequate transition zone material is also available. Routine tests of core and transition zone materials have been performed in Brazilian laboratories and have determined the relative stability of the material in the various borrow pits. In addition, test strips have been constructed at the site to correlate field and laboratory compaction results.

Coarse aggregate for concrete will be manufactured from quarried material at areas within reasonable hauling distance from the project. Rock at the project site generally breaks into slabs and blocks due to its hard, brittle nature, and the presence of bedding planes and joints. It can be used to manufacture aggregate, but such aggregates would produce harsh concrete. Limited quantities of sand are available in the stream bed of the Furnas River and tributaries for concrete.

DAM AND RESERVOIR

The preliminary investigations and report considered a number of combinations for development of the Furnas reach of the Rio Grande. Economic studies justified the highest dam that could be considered. Maximum elevation was limited by low saddles in two rims in the upper reaches of the reservoir rather than by physical conditions at the site. Normal pool level was set at elevation 766 m, and at this elevation one saddle dam is required to prevent the reservoir from spilling over into the Sao Francisco River basin. The maximum gross head will be slightly less than 100 m (328 ft) with the exact value being dependent on the volume of tailrace excavation.

At the normal pool level, a lake will extend approximately 217 km (135 miles) up the Rio Grande and 190 km (120 miles) up the Rio Sapucai. Gross reservoir capacity will exceed 21 billion cu m (about 17 million acre-ft). Usable storage in a 16-m (52.5 ft) drawdown exceeds 14 billion cu m (about 11.5 million acre-ft). The storage is adequate to control completely the run-off of the Rio Grande better than 95% of the time.

In early planning, consideration was given to limiting the maximum level to El. 750 m. Economic considerations proved the inadvisability of restricting the water level to this lower elevation because Furnas would lose 17% of its gross head, and 75% of its gross storage. Since there are no major storage sites above Furnas, loss of the reservoir capacity could not be overcome by a series of low dams that would develop all of the head.

The land to be inundated by Furnas reservoir is largely undeveloped. There will be some major railroad and highway relocations, and a few small communities will have to be re-established. Also, some good farm land will be lost to future production.

POWER CONSIDERATION

Since there is a market for the power as soon as it can be made available, it was important to realize optimum development of the resource at the earliest possible date. Although an extensive transmission grid does not exist at present, it is reasonable to assume that Furnas project will be interconnected to all of the major projects and load centers within the Belo Horizonte - Sao Paulo - Rio de Janeiro triangle. It can likewise be assumed that Estreito will be built in the near future so that Peixoto and Estreito will be both hydraulically and electrically integrated with Furnas. Existing and possible future upstream projects will have only minor effect on the power supply and negligible effect on reservoir operation as compared with Furnas and can be ignored in any power study for the Furnas site.

Power potential of Furnas was determined by making a number of studies for the 29-yr period of record. First computations were performed manually, but final studies were accomplished with an electronic computer. The minimum system which was considered included Peixoto and Estreito. Studies also were made assuming auxiliary sources of power in addition to the above projects. As might be expected for a project with a reservoir having a high degree of stream flow control, it was found that Furnas could economically utilize almost all of the flow.

Annual load factors of the existing utilities are in the range of 58 to 65%. Computer studies were based upon load factors of both 0.50 and 0.60; the former to consider possible future conditions, the latter to represent more closely the existing load. It was found that the project could economically generate an average of about 5.4 billion kwh annually and would waste less than 1% of the water. The installed capacity would be limited to about 1,200 mw, which is adequate to serve the project's share of the average annual system load at system annual load factor. It is recognized that a project of this magnitude should be considered for possible future peaking operations at a relatively low load factor.

The owners decided, after due consideration, that funds required now for provisions for possible future capacity expansion would be more effectively utilized for additional energy producing plants. No provision, therefore, has been made for more than eight units at this project.

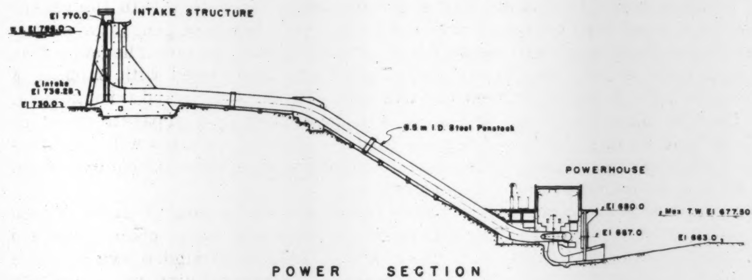
PRIME MOVERS AND GENERATING EQUIPMENT

Consideration of plant generation output and costs of various sizes of units led to adoption of a plant containing eight units, each capable of developing 150 mw, giving a total plant capacity of 1,200 mw. Present plans call for the first unit to be in service in mid-1962, with units two through four added at three-month intervals. No dates have been fixed for installation of the last four units, but it can be expected that they will be placed in service shortly following the initial installation.

Turbines were specified to deliver generator nameplate at near best gate at heads expected to prevail a large percentage of the time. Since the reservoir has a relatively small total drawdown which is utilized only in the driest years, the weighted average head is very high. This condition satisfied the Furnas Company's requirement that turbines be selected so they could not overload the generators by any appreciable degree (that is, not more than 6% to 8% at highest head). There results from this selection a small deficiency in capability under minimum head conditions. Since such a condition occurs infrequently (twice in the period of record) and then for only short intervals, this limitation was not considered serious by the owner. The turbines will have a nominal expected output of 192,500 metric horsepower (190,000 hp) when operating at a net head of 94 m (308 ft) at the point of best efficiency.

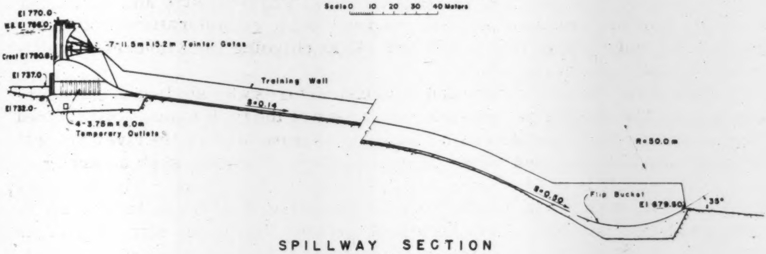
POWERHOUSE AND SWITCHYARD

The powerhouse shown in Figs. 2 and 3 will have an indoor type arrangement. Rainfall in Brazil during the wet season frequently exceeds 10 in. per


El 770.0
INTAKE STRUCTURE
W.S. El 766.0
℄ Intake El 736.25
El 730.0
6.5 m I.D. Steel Penstock
POWERHOUSE
El 680.0
Max T.W. El 677.50
El 667.0
El 663.0

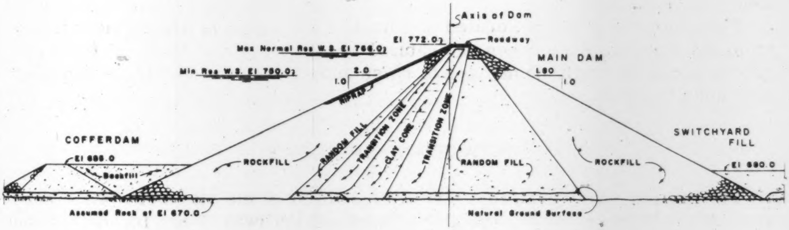
POWER SECTION

Scale 0 10 20 30 40 Meters


El 770.0
W.S. El 766.0
-7-11.5m × 13.2m Tainter Gates
Crest El 750.0
El 737.0
Training Wall
S=0.14
El 732.0
4-3.75m × 6.0m Temporary Outlets
R=50.0m
S=0.50
Flip Bucket
El 679.50
35°

SPILLWAY SECTION

Scale 0 10 20 30 40 Meters


Axis of Dam
El 772.0
Roadway
Max Normal Res W.S. El 766.0
MAIN DAM
Min Res W.S. El 750.0
2.0
1.0
1.80
1.0
RIPRAP
COFFERDAM
El 686.0
Backfill
ROCKFILL
RANDOM FILL
TRANSITION ZONE
CLAY CORE
TRANSITION ZONE
RANDOM FILL
ROCKFILL
SWITCHYARD FILL
El 690.0
Assumed Rock at El 670.0
Natural Ground Surface

DAM SECTION

Scale 0 20 40 Meters

FIG. 3.—FURNAS PROJECT SECTIONS

month for a period of five or six months. If an outdoor type plant were used, it would be necessary to provide shelter for dismantling and maintenance of the units, untanking of transformers, etc.

Tailrace deck level is set well above maximum tailwater, with the generator floor depressed to reduce height of building and length of generator shafts. The transformer deck will be on the upstream side of the powerhouse. This arrangement simplifies the structural problems associated with support of these pieces of heavy equipment and fits well with the switchyard arrangement.

Each turbine, as shown in Fig. 3, will be served by a separate penstock 6.5 m (21.3 ft) in diameter. Adequate load and speed regulation will be possible without surge tanks. It is expected that an acceptable amount of extra W R^2 will be provided in the generators.

It was decided to use single phase transformers, a bank of three to each generator. Each transformer will be of the forced oil water cooled type and rated at 53,333 kva. Bank connections will be delta to grounded wye with line to line voltages 15 kv and 345 kv respectively. Physical size and simplicity of replacement and transport limitations were basic considerations in this selection. The units will be tied to 138 and 345 kv outgoing lines through a multiple bus switchyard arrangement.

Orientation of the powerhouse and intake structures was studied at considerable length. The dip, strike and cleavage pattern of the rock foundation required these structures to be located with their long axes parallel to the river channel for maximum economy and safety against sliding. However, such an arrangement would result in undesirable flow conditions to the intake. The arrangement selected, as shown in Fig. 2, was a compromise of all the factors involved.

To make maximum use of available head between the Furnas site and Peixoto reservoir, an economic study was performed to determine the extent of tailrace excavation. It was found that a major amount of excavation could be justified.

The switchyard will be located on Ilha do Sapo, which is across the tailrace from the powerhouse between two natural river channels. Material from required excavation will be utilized to raise and enlarge the Ilha to its required size and elevation.

INTAKE AND SPILLWAY

The limited space available and configuration of the site were basic considerations in the arrangement of the intake and spillway. When foundation and materials investigations supported selection of a fill type dam, the locations for the intake, spillway, and powerhouse were automatically fixed. That is, they should go on the left side of the channel. Conditions for this layout are not ideal; neither are they seriously deficient. Foundation materials on the left were far better than those at the right abutment, but as noted above, dip and strike of the rock somewhat limited powerhouse orientation. Sound rock elevation is fairly low in a relatively small area, but it is adequate for spillway and intake structures.

The spillway occupies the extreme left section of the site. In addition to the aforementioned gated weir, it will have a concrete-lined chute with a flip bucket at the lower end. Water will be discharged into the natural river channel well downstream from the powerhouse. A short concrete non-overflow section will connect the right end of the spillway to the powerhouse intake structure.

With the arrangement as planned, flow conditions other than ideal were to be expected. These were studied in a 1:100 scale model of the intake and spillway area for a complete range of reservoir elevations. As a result of the model studies which were performed in Brazil, certain modifications to the original design proved necessary.

The spillway design flood was developed from the known hydrologic and meteorologic condition discussed briefly earlier in this paper. It was concluded that the flood would have a peak inflow of 18,000 cms (635,000 cfs). Volume being of prime consideration, this flood would have 128,960 cms-days in the first 31 days, 231,280 cms-days in the next 28 days, and 183,520 cms-days in the last 31 days. Flood routing studies were performed using different assumptions and it was found that discharges less than 10,000 cms (353,000 cfs) would pass with no reservoir surcharge. This was true even though the reservoir was assumed to be at full elevation 766.0 m when the flood began.

The spillway will be designed to discharge 1300 cms (460,000 cfs) with the reservoir surcharged 3 m to El. 769.0 m. Discharges will be controlled by seven radial crest gates each 11.5 m (38 ft) wide by 15.2 m (50 ft) high. The gates will be provided with individual hoists. In addition, the intake gantry crane will be able to handle spillway stop logs and heavy lifts during maintenance operations.

The intake and spillway decks were set at El. 770.0 m to provide an extra meter of freeboard should the reservoir ever be surcharged to El. 679.0 m. The crest of the rockfill dam was fixed at El. 772.0 m to provide additional safety against overtopping.

The intake structure is designed as a concrete gravity dam with one monolithic block per penstock. An extensive drainage system of down holes with a drainage tunnel will be provided in the quartzite foundation to prevent excessive seepage pressures developing downstream of the intake and spillway crest structure. There will be conventional trash racks, with provision for raking and trash removal to be installed later if required. Gates will be individually cable hoist operated. A separate gantry crane will be provided for heavy lifts and raking operations. The right end of the intake structure will abut the quartzite cut slope at the left end of the main dam. Both intake structure and spillway decks will be wide enough for vehicular traffic.

It is expected that little grouting will be required under the spillway, non-overflow and intake structure concrete. Water tests in exploratory drill holes have shown the quartzite to be relatively tight. This has been verified further by conditions encountered during excavation of the diversion tunnels.

DAM

The canyon at the Furnas site is 350 m (1,150 ft) wide on the axis at river level (Fig. 4). Natural ground extends well above maximum reservoir level on both abutments. Opposed to these favorable conditions the foundation in the river bed is traversed by a deep fault that has been closed with a diabase intrusion.

The section of the dam as shown in Fig. 3 was adopted after both circular arc and sliding wedge analyses showed that it was a structurally stable design. A machine language computer program was used to aid these studies. Strength parameters for all materials were obtained from laboratory tests performed in Brazil.

The core and transition zones are arranged as a compromise between an inclined core and a central vertical core. This arrangement was found to utilize available materials most effectively. The overburden material from the spillway excavation was found to be suitable for use in the transition zone to prevent particle migration from the core. Rockfill will be placed in layers and compacted by crawler-type equipment.

Horizontal curvature of the main embankment has been accentuated to improve abutment contacts and minimize the possibility of cracks developing in the core due to downstream movement. Because of the steepness of the left abutment (Fig. 5), special precautions will be taken to ensure that detrimental effects of differential settlement will not endanger the safety of the core. The upstream and downstream transition zones will be extended along the rock face.

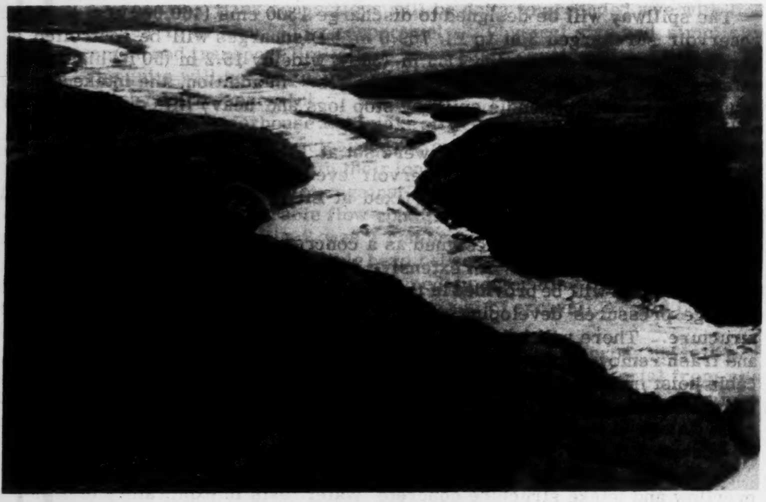


FIG. 4.—SITE OF FURNAS PROJECT

Maximum height of the dam above foundation excavation will be about 120 m (395 ft). Exact elevation of deepest excavation will not be known until the main channel is unwatered. However, core drilling through the diabase dike indicates the material is sound, with the possible exception of fractured and decomposed material immediately adjacent to the diabase. Unsuitable material will be removed and replaced with impervious fill. Limited grouting is planned and will consist principally of test holes to check watertightness and grout take. Excavation to suitable foundation on the left abutment is expected to be nominal. The right side will require considerably more excavation to provide a suitable contact area for the clay core against sound impervious abutment rock.

Total embankment material is expected to be about 9,000,000 cu m (roughly 12,000,000 cu yd). About 600,000 cu m (800,000 cu yd) will be clay material for the core and cutoff. Ample borrow area for this material is located on the left bank about 1 mile upstream of the dam. Transition zones will require 1,350,000 cu m (1,640,000 cu yd), sound rock 4,300,000 cu m (5,400,000 cu yd) and random fill about 2,275,000 cu m (3,000,000 cu yd). Required excavation will provide all main dam embankment materials except those necessary for the clay core. Materials from excavation unsuitable for the main dam will become a part of the switchyard fill on Ilha do Sapo or will be wasted in the river channel at the right of the island.

STATUS OF PROJECT

The major construction contract was awarded in mid-1958 to a joint venture. Work was started on the two 12.8 m x 13.5 m (42 ft x 44 ft) horseshoe-shaped diversion tunnels, driving by the heading and bench method. The bench was "holed through" in the latter part of June, 1959. Rock is generally excellent, though in a limited reach near the upstream portals schistose material was encountered, requiring support. At present the arch is being lined by use of the "Aliva" process, a European equivalent to shotcrete, with mesh reinforcing used as required.

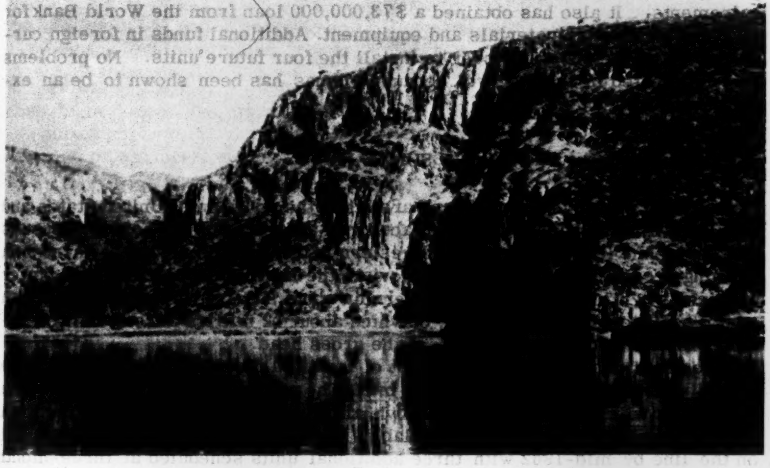


FIG. 5.—LEFT ABUTMENT, VICINITY OF DIVERSION TUNNEL INTAKE PORTALS

Cofferdams confining the river into the deep central channel have been built on both sides of the river, with considerable excavation and scaling at each abutment. Present plans call for diversion through a channel on the right side during the next season, with the tunnels carrying only a portion of the diversion flow. Later, when the channel is closed and fill is placed on the right side, all flow will be diverted through the tunnels. Tunnels will be plugged after closure bulkheads are lowered and the reservoir starts to fill.

Due to its remote location, it was necessary to build housing and allied facilities at the site. This work, along with mobilization of men and equipment, required an extensive period of time and was partially completed by the owner prior to award of the construction contract. The owner will award separate supply contracts for furnishing all major items such as electrical and mechanical equipment, gates, cranes, hoists, special steel items such as penstocks and trash racks, gates, switchyard equipment, etc. These items will be furnished where possible by Brazilian manufacturers. Major items not available in Brazil will be imported.

COST AND FINANCING

Total project construction cost with four units installed, exclusive of transmission lines, is currently estimated to be $129,000,000 with an additional $40,000,000 required to bring the plant to its ultimate capacity. The Brazilian government, as principal sponsor, is furnishing the bulk of local currency requirements. It also has obtained a $73,000,000 loan from the World Bank for purchase of foreign materials and equipment. Additional funds in foreign currency will be required in order to install the four future units. No problems are anticipated in this regard, because Furnas has been shown to be an excellent project.

SUMMARY

Brazil, a country larger in land area than the continental United States and populated by 65 million persons, is faced with a severe power shortage. Furnas project on the Rio Grande in Southwestern Minas Gerais is one element in a growing program that will help to alleviate the lack of adequate electric power and energy. This project has a reservoir about 240 km (150 miles) long with over 17 million acre-ft of water, of which about 12 million acre-ft will be usable with a drawdown of only 17% of the gross head.

The initial installation will consist of four 150 mw units, with the ultimate capacity of eight units totaling 1,200 mw. The average annual generation with eight units will be about 5.4 billion kwh, all of which will be usable in the area load as soon as it can be made available. Present plans contemplate first unit on the line by mid-1962 with three additional units scheduled at three-month intervals. No firm schedule has been set for the last four units, but they will undoubtedly be required at an early date.

The main dam will be a composite fill-type structure containing approximately 9,100,000 cu m (12,000,000 cu yd) of material. It will be some 580 m (1,900 ft) long at crest elevation and will have a maximum height from deepest excavation to crest level of about 120 m (400 ft). A spillway and chute on the left abutment will pass nearly 10,000 cms (350,000 cfs) at normal full pool level and 13,000 cms (460,000 cfs) with a 3-m (10-ft) surcharge.

Although there is only one existing downstream project, potentially there are sites for about a dozen more. The firming effect of Furnas reservoir on these future projects will enhance its regional value considerably. Furnas would be considered a large project in any country. For Brazil it represents a tremendous undertaking, and it is a tribute to the courage and efforts of her people. Although they are utilizing foreign consultants, the bulk of the design work is being performed by Brazilians.

PERSONNEL

Central Eletrica de Furnas is headed by John R. Cotrim, M. ASCE, president, Flavio H. Lyra, technical director, B. Dutra, financial director and F. Von Ranke, chief engineer. Cia. Internacional de Engenharia e Construcoes, a Brazilian company associated with International Engineering Company, Inc., is preparing the construction drawings and specifications. The technical staff of Sao Paulo Light is available for consulting service. Other consultants include A. Casagrande and S. D. Wilson for soils; Portland P. Fox, F. ASCE, geologist; E. P. Eardley for transmission facilities; R. L. Hearn of Ontario, Hydro. Geological data given in this paper are based on the work of J. Cabrera, staff geologist of Sao Paulo Light. Meroz and McLellan of Newcastle upon Tyne, England, and Ebasco Services, Inc., were engaged by the client on related problems. Hydraulic model work was performed by the Escritorio Saturnino de Brito Hydraulic Laboratory in Rio de Janeiro.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3136

DEVELOPMENT OF ROTATIONAL IRRIGATION IN TAIWAN

By Lee Chow,[1] F. ASCE

SYNOPSIS

Rotational irrigation, now rapidly replacing the old irrigation practice in Taiwan, is described with special emphasis on design and operation of such irrigation systems. Some background information on rotational irrigation experimentation and demonstration is given. Future improvements to be made and possible research to be done are also pointed out.

EARLY CONTINUOUS IRRIGATION PRACTICE

The irrigation history in Taiwan dates back to Yuan Dynasty (1277-1367) in Chinese history. People built canals to irrigate the land they cultivated. The population and the farm land than were both scarce. There was no need to limit the use of irrigation water. That was the origin of the old continuous irrigation practice.

Rice plantations are the most common in Taiwan. Wherever irrigation water is adequate, two rice crops are grown in a year. Before rice transplanting, the field is soaked with water. For sandy soil the soaking may be eliminated. After soaking, levelling of the ground will be done in preparation for transplanting. During the early growing season, there are two or three times each for weeding and application of fertilizer. The growing period of the rice crop is about

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Note.—Published essentially as printed here, in September, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2588. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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120 days and that of second crop is a little shorter. Excluding the time of weeding and fertilizer application and a short period before harvesting when no water is applied, irrigation continues for about 90 to 100 days.

FIRST EXPERIMENTS ON ROTATIONAL IRRIGATION

While with the Joint Commission On Rural Reconstruction (JCRR), the author was surprised at the low duty of irrigation water in Taiwan, which averaged about 400 hectares of paddy field per cu m per sec of water. This low duty encouraged the study of the local irrigation method. It was found that during 1933-43, extensive experiments had been performed but no collective study of the results had been made. Some experiments were intended to show the influence of different irrigation methods on crop yields. Some were to show the effect of different depths of water. Still others were to determine the best interval between irrigations. The experiments were extensive and the results were interesting. A total of 287 tests were studied, of which 198 were used to compare the crop yields by continuous versus rotational irrigation methods. About one half of the cases showed better yields for rotational irrigation. Of the 198 tests, 142 cases showed yield differences of less than 10% based on the yield by the local continuous irrigation practice, 43 showed differences of 10% to 20%, 9 showed differences of 20% to 30%, 1 showed a difference of 30% to 40%, 2 showed differences of 40% to 50%, and 1 showed a difference of over 50%.

The different irrigation intervals varied from 1 to 15 days. The 3 day interval, or 1 irrigation every 3 days, indicated a higher possibility of better yield than other intervals.

The different depths of irrigation water used in these tests varied from just enough water to prevent the soil from cracking to enough water to keep the field saturated. The intermediate water depths were 0.015 m, 0.03 m, 0.036 m, 0.045 m, 0.06 m, and 0.09 m. For further details, the reader is referred to the Engineering Series No. 3 of the JCRR.[2] From these tests, three conclusions were drawn:

DEMONSTRATION OF ROTATIONAL IRRIGATION

Since the result of the preceding study[2] was made known in 1953, the government and the people began to realize the importance of the problem, especially under the pressure of the over 3% annual increase of population. In 1954, the government established the Rotational Irrigation Promotion Commission (RIPC), the purposes of which were to promote rotational irrigation and to plan and supervise experimental and demonstration farms. Three experimentation farms were successively established. In each farm, a small area was set aside and subdivided into plots for various irrigation treatments, for example, rotational irrigation on 6-day, 8-day, and 10-day intervals and continuous application. Several replications were provided for each different treatment. The amount of irrigation, precipitation, outflow from surplus rainfall, evaporation, sunshine, etc., were all carefully recorded. There were over 10 demonstration

1. As far as the crop yield is concerned, there is no appreciable difference between the two methods;
2. most tests showed appreciable decrease in water depth for the rotational irrigation method; and
3. the tests showed a water saving of 15.8% to 38 7%, with 26.1% as the average for the rotational method.

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[2] "Rotational Versus Continuous Irrigation Methods For Taiwan," by Lee Chow, Engrg. Series No. 3, Joint Comm. on Rural Reconstruction, 1953.

farms throughout Taiwan. On these demonstration farms, the quantity of water, though also measured, is not very strictly controlled. The farms, on one hand, were to show the results by rotational irrigation as compared with the continuous practice, and on the other hand, to exploit actual difficulties when rotational irrigation is to be put in practice on a large scale. For a more detailed description of the experimentation farms, the readers are referred to another paper.[3]

Records, starting in 1955, are available for the experimentation and demonstration farms. Although there was some inconsistency of results due to too many varying factors, two points are clearly verified in a great majority of cases. One is that rotational irrigation does give higher yields and the other is that the amount of water saving is at least 20% to 30%, and sometimes it is as high as 50%.

Rotational irrigation was put into use for the first time on a large scale and its merit was fully recognized in the 1954-55 drought when the rivers and canals had only 1/3 to 1/2 of the ordinary flows. Rotational irrigation had achieved remarkable results. A total of 35 out of 40 irrigation associations practiced rotational irrigation and had a total planted area of 195,959 hectares. If rotational irrigation had not been practised, the transplanted area would have been only 139,797 hectares, or an increase of 51.7% was achieved by rotational irrigation.

Two important actions were taken by the government to help enforcement of rotational irrigation. One was the promulgation of an irrigation regulation in February, 1955. This regulation has special reference to rotational irrigation. The main purpose of it was to legalize the features involved in implementation of rotational irrigation. This was the first time that irrigation in Taiwan had regulation. The second was the authorization of a four-year plan in which a total of 112,808 hectares of paddy field are to be provided with new, or strengthened with improved, irrigation systems in order to enforce rotational irrigation.

DESIGN OF ROTATIONAL IRRIGATION SYSTEM

Different Methods of Rotational Irrigation. **—Rotational application of water may be done in the following three ways:**

A. Rotation by sections in the main canal,
B. rotation by sections in the laterals or sublaterals; or
C. rotation by farm ditches.

In case A, the water will be conveyed in turn to different sections of the main canal. If the main canal is divided into three sections, the water will be conveyed in turn to the 1st, 2nd, and 3rd sections. In case B, rotation by sections in the laterals or sublaterals, the main canal will have continuous flow while the water will be conveyed to the various sections of laterals or sublaterals successively. In case C, rotation by farm ditch, the water flow in the ditch will be intermittent while the flow in the main, laterals and sublaterals will be continuous. In this case, the whole area is divided into rotation areas. There may be any number of rotation areas. The use of water is rotated in the subdivisions of a rotation area, called rotation units. For example, a certain rotation area is divided into 5 rotation units, and the rotation interval is 5 days,

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[3] "Rotational Irrigation—An Evolution in Taiwan," by Lee Chow, Newsletter of Internatl. Rice Comm., U. N., March, 1958.

each rotation unit will get its share of time of irrigation application in proportion to its area. The sum of all application periods of the rotation units in a rotation area will be the rotation interval.

In the afore mentioned three types of rotation, experience has shown that case C is the best because type A will call for the same capacity for the whole main canal, type B will call for the same capacity for the whole laterals or sublaterals, while case C will have diminishing capacities and call for larger farm ditches. Moreover, case C can be better incorporated into the practice of common irrigators, which will be explained later, than the other 2 types and will not call for enlargement of existing canals when used for rotational irrigation.

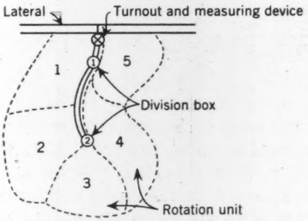
Lateral
Turnout and measuring device
1
5
①
Division box
2
②
4
3
Rotation unit

FIG. 1.—ROTATION AREA AND ROTATION UNIT

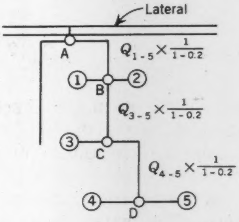
Lateral
A
$Q_{1-5} \times \frac{1}{1-0.2}$
①
B
②
$Q_{3-5} \times \frac{1}{1-0.2}$
③
C
$Q_{4-5} \times \frac{1}{1-0.2}$
④
D
⑤

FIG. 2.—DERMINATION OF Q

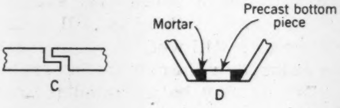
A
B
C
Mortar
Precast bottom piece
D

FIG. 3.—PRECAST CONCRETE SLAB FOR LINING

Rotation Area and Rotation Unit.—In the design of a rotational irrigation system, the first step is to determine the size and location of the rotation areas. There is no fixed rule for the best size. The layout should be planned according to topography, existing roads and water courses, nature of soil, and other ground features. Experience has shown that the size of 50 hectares is on the average a good rotation area size. It should be noted, in general, that a rotation area should not be too large to require a discharge in the rotation area canal which cause erosion. Also a rotation area should not be too small as to call for a very small amount of flow because, in case of a very long ditch, there would be serious difficulty in flow conveyance. Fig. 1 shows the general idea of how the rotation area and the rotation units are laid out. The symbol X denotes the turnout structure and the measuring device, "1" denotes a division box from which water is delivered in turn to rotation units 1 and 5. Division box "2" denotes another division box where water is delivered in turn to rotation units 2, 3, and 4. If the rotation interval is 5 days, and, for the sake of simplicity, the rotation units are all of the same size, it would result that all the 5 rotation units will have equal share of time, that is, 1 day, In case of discharge fluctuation, the irrigation schedule should then be changed to fit that particular situation. If there is precipitation during irrigation, the schedule is also changed to save water.

Determination of Canal Capacity.—The proper way to determine the canal capacity is to start from the lower end and work backwards. The following formula has been used:

$$Q = \frac{A}{8.64}\left(\frac{d_s}{p_s} + \frac{d_r}{p_r}\right)\frac{1}{1 - L} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which Q is the required discharge in cu m per sec, A is the irrigated area in hectares, d_s is the depth of water in meters required for soaking the field, varying from 0.12 m to 0.15 m in the local farming practice, d_r is the depth of water in meters for each application, p_s is the period of soaking the field in days, p_r is the rotation interval in days, and L is the canal conveyance loss in percentage. The water depth d_r varies with the crop, the nature of soil, and other factors. In the rotational irrigation schemes, in Taiwan, the water depth used for each application varied from 0.03 m to 0.06 m. The rotation interval p_r varied from 2 to 8 days, with the 6-day interval being most common. The soaking period, p_s, varies with the soil structure and it is usually around 20 days. For sandy soil, soaking may not be required. It has been found that p_s could be shortened to less than 20 days, whereby further water saving could be effected. The rotation interval, p_r, after having been determined, is to be closely investigated and adjusted, if necessary, during irrigation operation. At the end of the rotation interval, if there is still some water left from previous application, or the field is still wet, it would mean that either the depth of irrigation ought to be reduced or the rotation interval ought to be lengthened. The canal conveyance loss, L, may be assumed at first by the designer and should be checked by the irrigation operators and modified to conform with the actual conveyance loss.

Fig. 2, shows five rotation areas. The discharges required for each area are calculated first using Eq. 1 without the loss factor 1/1-L. The Q between C and D will be Q_{4-5} times the loss factor. The Q between B and C will be Q_{3-5} times the loss factor. The Q required between A and B will be $Q_{1-5}\left(\frac{1}{1-L}\right)$.

Structures and Measurements.—The design of the structures in a rotational irrigation system is not different from that in other types of irrigation systems. The following 5 items are examined with the hope of casting some light on future improvement.

A. Turnout gates.—Either in new rotational irrigation systems or in remodelling old systems, turnout gates are the most common installation. Without these, the water for a rotation area cannot be regulated or controlled. It has been

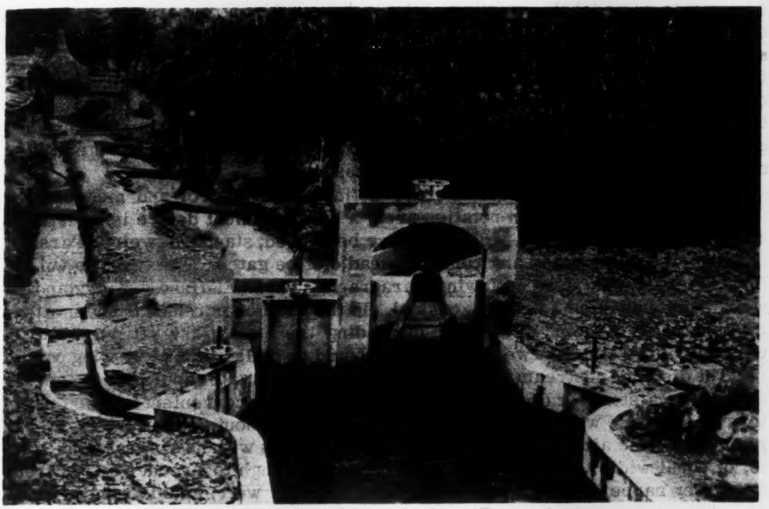

FIG. 4.—TURNOUT GATES

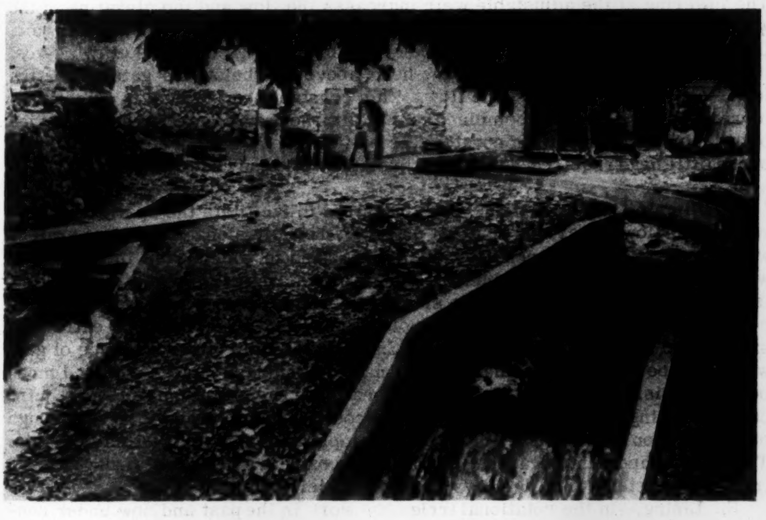

FIG. 5.—TURNOUT GATE WITH RECTANGULAR WEIR BELOW

common to remodel the existing systems by combining several laterals into one. These laterals previously tapped water directly from the main canal. In any case, each rotation area should be provided with a turnout gate with a locking device. Several forms of lock are being manufactured in Taiwan. The best one is a combination lock incorporated in the gate hoist. In case the combination number is revealed, the lock can be replaced easily. A standard turnout gate with wedges has been very satisfactorily used, specially for water tightness. Figs. 4 and 5 show such turnout gates.

B. Measurement Device.—In order definitely to control and measure the flow delivered to a certain rotation area, a measurement device is indispensable. Various measurement devices have been used; standard weirs, Parshall flumes, adjustable gates and constant-head orifice gates. The last two will be further explained in the following paragraph. Parshall flumes and standard weirs have been found very successful. In the case of Parshall flumes, attention should be directed to the ease of reading the staff in the stilling well. In the case of the standard weirs, attention should be called to the following points: (1) The distance upstream from the weir to the turnout gate should be long enough to produce a steady, calm flow but not loo long to be inconvenient for the operators who have to walk back and forth between the turnout and the weir for adjusting; (2) the air vent pipe required for the weirs without contractions, that is, of equal width to the canal, should be large enough to produce a true free-fall water nappe. A triangular weir, a rectangular weir, and Parshall flumes are shown, respectively, in Figs. 5, 6, and 7.

C. Combination of items A and C.—Sometimes it is more convenient to use a combination structure to combine the turnout gate and the measurement device. Adjustable weir serves as a turnout as well as a measurement device. The lowering of the adjustable weir increases the flow and the elevating of the weir decreases it. The flow is measured in the same way as in a standard weir. Constant head orifice is another type of combination. It consists of two gates; the upper one is set with a constant head above and below the gate, while the lower one is used to adjust the flow. This constant head orifice is sometimes more expensive than the combined cost of a turnout and a measurement device but is more convenient to operate. The orifice type is also preferable because of the little head required, while the adjustable weir type requires a much larger head.

D. Division box.—Division box is required to facilitate the operation of diverting the flow to various rotation units. Without division boxes, the operators would have to plug or open a farm ditch by hand in which case the operation would be cumbersome and time consuming. One drawback of the division box is that it usually leaks due to too small a pressure against the gate. It is important that the contacting surfaces of the gate and the grooves be extremely smooth and true in dimensions. T. R. Smith, devised an adjustable type of gate which can be operated in small increments of openings. It was not used much because division boxes as used in rotational irrigation systems seldom require partial openings and because of its higher cost. Any improvement on the design of the division box must be economical, simple in design, and easy to operate. These are important because many division boxes are required in any rotation irrigation system. Fig. 8 shows two such division boxes.

E. Lining.—In the rotational irrigation work in the past and now under construction, precast concrete slabs have been used to advantage. (Fig. 9) It is economical and does away with the difficulty of mixing small amounts of concrete in scattered places and, most important of all, it renders possible steeper

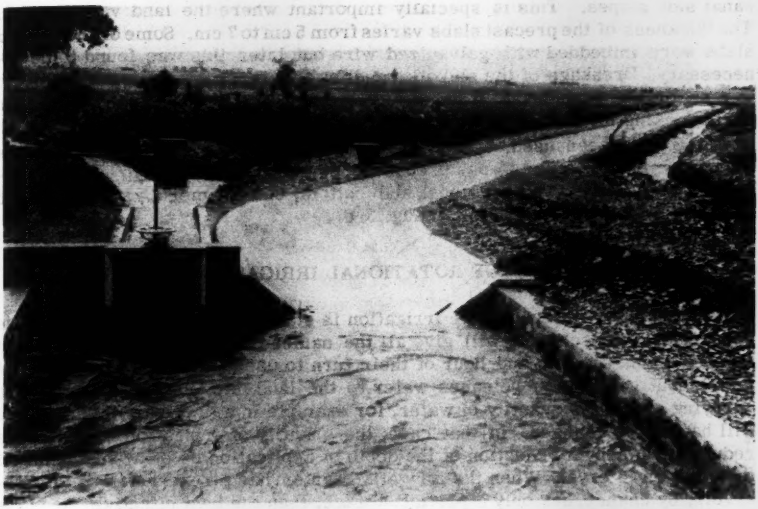

FIG. 6.—TRIANGULAR WEIR

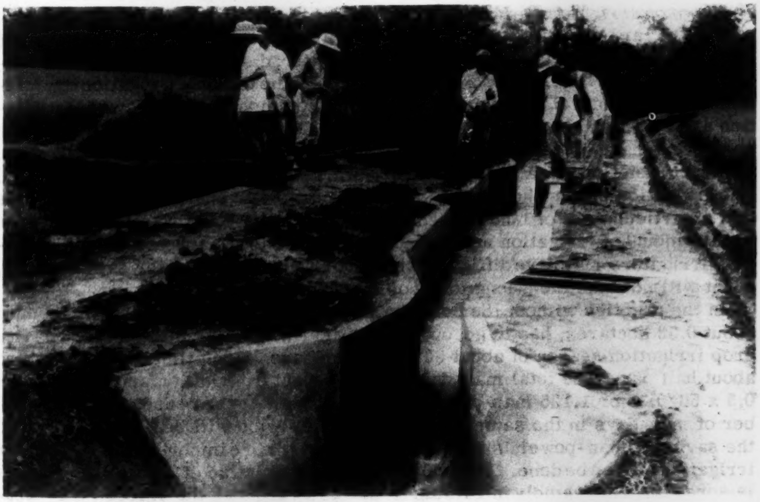

FIG. 7.—PARSHALL FLUMES

canal side slopes. This is specially important where the land value is high. The thickness of the precast slabs varies from 5 cm to 7 cm. Some of the earlier slabs were imbedded with galvanized wire but later this was found to be unnecessary. Breakage of the slabs in handling and transportation can be reduced to a negligible percentage, if handled with care. It is to be noted that a firm earth backing for the slabs is absolutely necessary. Various joints as shown in Fig. 3 have been used. The type in Fig. 3(b) proved to be the best. It is also a common practice to use precast slabs for the sides and to cast the bottom in place. In case of a precast slab bottom, cement mortar should be used to fill up the corners as shown in Fig. 3(d).

OPERATION OF ROTATIONAL IRRIGATION SYSTEM

Irrigation Schedule.—Before irrigation is started, an irrigation schedule is prepared. This schedule will give all the names of the water users, the land number and area, date and hour of their turn to use the water, the number of the turnout gate which releases water to the land, etc. In case the canal is carrying a smaller quantity of water, for example in a dry year, this schedule will have to be changed. In that case, the quantity of irrigation water will be reduced either by a reduction of the depth of application or by a lengthening of the rotation interval. When the irrigation system has a reservoir and the reservoir, by chance, has only a partial storage, then the best effective use of the available water supply can be easily planned. This schedule is posted and made known to the water users. In Taiwan, this operation is done by the irrigation associations. There is a total of 40 irrigation associations which have the responsibility of operation and maintenance of all the irrigation systems.

Common Irrigators.—In the earlier period of practicing the rotational irrigation, the farmers, during their turn of applying irrigation water, had to come out to their fields to take care of diverting water, distributing water, maintaining ditches, etc. Later on in southern Taiwan, in the Chianan Irrigation Association area, there developed a practice of using common irrigators within a rotation area. These common irrigators, selected by the farmers within the rotation area, were hard-working, impartial, honest and familiar with the whole area. For each rotation area of 50 hectares, two common irrigators are required. The irrigation of the whole area would be under their charge. They would see that a particular lot of land needed some adjustment in the rotation interval or in the amount of irrigation according to the soil or to its appearance. The common irrigators may have difficulty at first distributing the water satisfactorily, but it will not take them long to become experienced.

In the practice without the common irrigators, an average farmer, farming about 0.33 hectares, has to go out to his field about 15 times during one rice crop irrigation season of about 90 irrigation days. Each time he has to spend about half day. The total man-days for a 50-hectare rotation area will be 15 x 0.5 x 50/0.33 or 1,125 man-days. If common irrigators are used, the total number of man-days in the same area in the same period will be 90 x 2 ot 180. So the saving of man-power is appreciable. What is more important is that a better irrigation job can be done. This explains why the practice of common irrigators is spreading very rapidly in Taiwan. It naturally reduces the agricultural population. The people who previously were engaged in agriculture can now help in industrial or other development work.

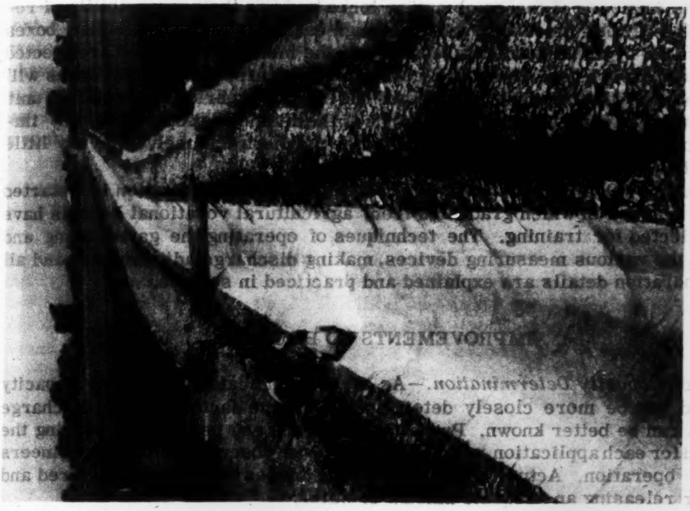

FIG. 9.—PRECAST CONCRETE SLAB LINING

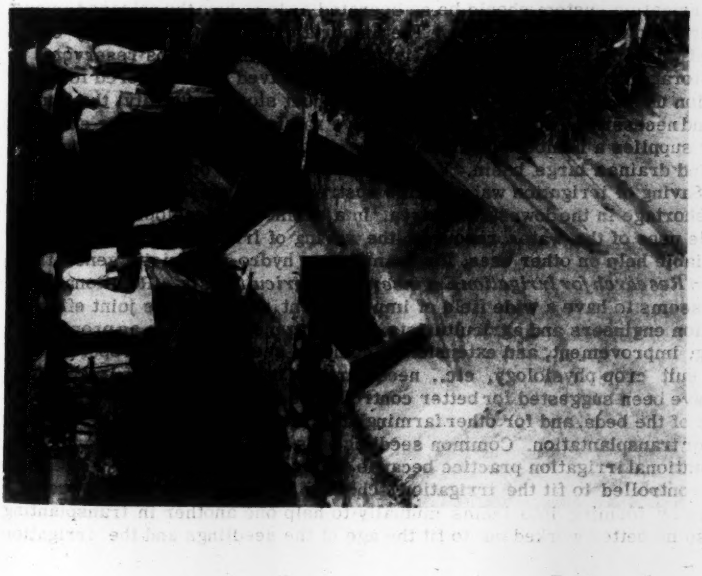

FIG. 8.—DIVISION BOXES

Operation of Gates.—The operation of the turnout gates is in the hands of an irrigation assistant employed by the irrigation association. He is the man who knows the numbers of the combination locks and has the responsibility of releasing the proper amount of water through each turnout. The division boxes are operated by an irrigation subcommittee the members of which are elected by the farmers. Where there are common irrigators, the division boxes will be operated by the common irrigators. The irrigation period each rotation unit is entitled to have is clearly in the minds of the division box operator. Any improper operation will be self checked by the farmers themselves. Very little dispute has been reported.

Training of Operators.—Since 1956, each irrigation association has started training classes in which graduates from agricultural vocational schools have been selected for training. The techniques of operating the gates, using and reading the various measuring devices, making discharge adjustments, and all other operation details are explained and practiced in service.

IMPROVEMENTS TO BE MADE

Closer Capacity Determination.—As already indicated, the canal capacity required can be more closely determined if all the factors in the discharge formula can be better known. Proper water depths and periods for soaking the field and for each application would depend on close observation of the engineers in canal operation. Actual canal conveyance losses should be measured and used for releasing an adequate amount of water.

Better Operation for More Effective Rainfall.—The higher the effective rainfall, the less the artificial irrigation. Whenever there is precipitation, the gates of the irrigation system should be so operated as to reduce the released amount of water or to cut it out entirely. The importance of this can be very easily realized and appreciated when the irrigation system contains reservoirs or other storage facilities because the water thus saved can be stored for later irrigation use. Even where the system is without storage facility, the importance and necessity of increasing the effective rainfall still exist. For example, a river supplies a number of irrigation system and the river covers a long distance and drains a large basin. Very often the rainfall only covers a small area. Saving of irrigation water on the upstream area may help greatly on the water shortage in the downstream area. In a basin-wide development involving multiple uses of the water resource, the saving of irrigation water may be of appreciable help on other uses, for example, on hydro-electricity generation.

Joint Research for Irrigation Engineers and Agriculturists.—Rotational irrigation seems to have a wide field of improvement requiring the joint effort of irrigation engineers and agriculturists. Farming methods, such as proper field soaking, improvement, and extension of common seedling beds, proper irrigation to suit crop physiology, etc., need further research. Common seedling beds have been suggested for better control on seed treatment, for better preparation of the beds, and for other farming practice such as proper age of seedlings for transplantation. Common seedling beds can very well be incorporated into rotational irrigation practice because the age of the seedlings can be perfectly controlled to fit the irrigation schedule. The cooperation between the farmers by forming into teams mutually to help one another in transplanting can also be better worked out to fit the age of the seedlings and the irrigation schedule. To transplant the seedlings at the proper time is very important in rice culture. It would be very interesting if some research program work laid out trying to devise an irrigation pattern to conform with the water requirement and water absorption capacity of various plants on the basis of plant physiology. Rice has been found in the Taiwan Agriculture Experiment Station to have a definite period in its growth in which the capacity to absorb water is the highest and another period in which the rice crop demands drainage. There may exist some special pattern of irrigation which would require the least amount of water and yet would give the highest yield.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3137

GEOPHYSICAL PROCEDURES IN GROUND WATER STUDY

By H. R. McDonald,[1] M. ASCE and Dart Wantland[2]

SYNOPSIS

Although geophysical methods of subsurface exploration were developed primarily for, and have been used most extensively by, the petroleum industry, many of the procedures apply equally well in the search for ground water. The principles of the geophysical methods most useful in ground water exploration are described and illustrated. In addition, a new application of the electrical bore-hole logging is described which may be of great importance in locating leaky zones in irrigation canals.

INTRODUCTION

Most engineers engaged in ground-water investigations have some general knowledge of geophysical methods. However, in many cases, the use of potentially valuable geophysical tools is overlooked because what they can do to help locate suitable aquifers is not generally understood.

For this reason, the purposes of this paper are as follows:

1. To explain briefly the principles of those geophysical methods most useful in ground-water investigations;
2. To illustrate the use of these methods by examples;
3. To discuss a new development in geophysics which may have application to irrigation and drainage problems.

Note.—Published essentially as printed here, in September, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2589. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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Although geophysical exploration was mainly developed, and has been most widely used in the search for oil, much of the knowledge thus gained, and many of the procedures employed apply equally well in the ground-water field.

THE GEOPHYSICAL METHODS

The water-well driller has different sets of tools with which to put down holes. He may use cable tools or a rotary rig or select a rock bit or a diamond bit as most suitable to a drilling problem. In like manner, the geophysicist has sets of tools or exploration procedures with which to probe beneath the surface. He may use one of four major geophysical methods—the seismic, the electrical, the magnetic, or the gravity method—whichever is most suitable in a given case.[3,4]

The Seismic Method.—In the seismic method, the speed of travel of seismic waves in different subsurface layers is measured. The waves are created by exploding small charges of dynamite buried in shallow holes. The depth to these layers and their thickness are also measured and such geophysical data can be correlated with definite horizons established by drilling. We know, for instance, that seismic waves travel faster in consolidated rock than in unconsolidated overburden. Therefore, a layer in which seismic waves travel at a high velocity may be bedrock in a buried channel. Mapping such a channel by seismic measurements, using the "refraction" procedure, may establish where water bearing gravel has the greatest thickness and, hence, the best place to drill.

The Electrical Method.—In the electrical method, most used in ground-water investigations, direct current is sent through the ground between two metal stakes. This permits the electrical resistance or resistivity of earth materials to be measured. Resistivity is a definite characteristic of rocks and formations and makes it possible to distinguish different types of materials from each other. For example, dry gravel has a higher resistivity than wet gravel and by way of contrast, clay and silt have a very low resistivity.

In the resistivity method, metal stakes, through which the electrical current enters the ground, are moved farther and farther apart in the course of making a set of measurements. As a result of this movement the current goes progressively deeper. In this way, the resistivity of a larger and larger volume of earth is measured and data are obtained with which to plot a resistivity depth curve. Such measurements have been called "vertical electrical drilling." Field results can be correlated at drill holes and the depth to horizons determined, as related to ground water.

The Magnetic Method.—In the magnetic method, a magnetometer is used to measure the vertical component of the earth's magnetic force at stations in an area. (The horizontal component of the earth's magnetic field is the one which causes a compass needle to point to the north.)

The magnetometer can be employed to study subsurface conditions because the magnetic properties of rocks are indicative, in many cases, and measurably affect the earth's magnetic field. For example, a basalt dike is almost invariably highly magnetic. A basalt dike, therefore, can often be traced by magnetic measurements, even though it is buried, because magnetic force is a maximum at points on the surface near to and just above it. Correlations can be established at drill holes and outcrops to guide the interpretation of magnetic surveys.

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[3] "Geophysical Exploration," by C. A. Heiland, Prentice Hall, Inc., New York, 1940.

[4] "Exploration Geophysics," by J. J. Jakosky, Times Mirror Press, Los Angeles, 1940, Revised Edition, Trija Publishing Co., Los Angeles, 1950.

Ground-water geologists are familiar with the "barrier action" of dikes which may dam up underground water and cause it to be impounded in adjacent pervious formations. In other cases, water may accumulate in fractured contact zones between dikes and neighboring impervious beds. The magnetic mapping of dikes is an accepted geophysical tool in ground-water exploration.

The Gravity Method.—In the gravity method, the force of gravity is measured at stations on traverses with a gravity meter. Gravity meter measurements supply subsurface information because the density of layers or rocks of different kinds, and marked contrasts in density, also affect the force of gravity in a local area. This is illustrated by the use of a gravity meter to trace buried dikes (which is possible because the dike rock has a high density), which causes an increase in gravity force near the dike. The gravity meter can also be used to show the configuration of a relatively high density bedrock surface along a buried channel or in a sedimentary basin. Both of these applications are useful in finding ground water. Correlations of gravity measurements can be made at drill holes and outcrops just as in magnetic surveys.

General Considerations.—The preceding indicates that all geophysical methods have three common characteristics. They require; (1) certain measurements to be made at the ground surface using special instruments, (2) that these measurements relate to, or reflect properties of, buried rocks and formations or structural conditions, and (3) that these relationships permit the geophysical data to be interpreted in terms of subsurface geology.

That geophysical findings of a survey should be correlated with drilling or outcrops has been referred to. Such drill hole correlation, by which the significance of subsurface horizons can be verified, lends strength and certainty to geophysical results.

Geophysical exploration can be defined as a type of field investigation where measurements are made at the ground surface with special instruments to secure subsurface geological information.

Basis of Application.—Knowledge of the geology of an area is essential to the understanding of the occurrence of ground water because geologic structure and stratigraphy provide the framework in which ground water, recharge, storage, and discharge take place. Therefore, any process that increases our knowledge of geologic conditions will help the investigator to determine the likelihood of the occurrence of ground water in a given area. It will help him to determine whether ground water may occur in small or in large quantities, whether the ground water is sweet or saline, and whether the geologic formations are likely to yield water to wells freely.

Inasmuch as the geophysical methods previously described will, under certain conditions, contribute to a better understanding of the geology of an area, they are useful tools in ground-water investigations.

The authors do not mean to imply that geophysical methods may or should replace other means of exploring for ground water. We wish to bring out that, in many cases, geophysics can be used quite advantageously to supplement the procedures usually employed for such purposes.

EXAMPLES OF GEOPHYSICAL INVESTIGATIONS

There are numerous examples of the successful employment of geophysical methods in locating ground water and/or the best place or places to drill in order to find ground water. These examples are described in technical articles scattered through a variety of publications. To specifically illustrate the manner in which geophysical field investigations were applied to particular ground-water problems, two examples from investigations made by the United States Bureau of Reclamation, Dept. of Interior (USBR) are presented.

Weber Basin Project.—The Weber Basin Project of the USBR is now (1960) under construction. It is designed to irrigate more than 54,000 acres of new land and supply supplemental water to some 24,000 acres now short of water. The project area contains the major concentration of population in the state of Utah, where municipal and industrial demands for water are growing rapidly. Even with the increased water to be supplied by the project works, there currently is need for additional water.

The area east of the shore of the Great Salt Lake is underlain by an artesian basin that extends over some 200 sq miles. Water from artesian aquifers supplies many farmsteads as well as several cities. In certain local areas, these aquifers contain slightly brackish water.

To test the ground water possibilities of the basin, two wells were drilled by the USBR in 1955, to depths of 1,220 ft and 977 ft, respectively, using cable tools. It was decided to explore for deeper aquifers, and, subsequently, a third well was drilled, using a rotary rig, to a depth of 3,006 ft. Prior to drilling it, however, seismic and gravity meter surveys were conducted to secure information on the depth, thickness, and continuity of aquifers and the configuration of the basement rock. These surveys were made to aid in selecting the best location for this deep test well.

Geologic conditions.—The "East Shore Area" in which the city of Ogden, Utah is located, is bounded on the west by the Great Salt Lake and on the east by the steep slopes of the Wasatch Mountains. These mountains are composed of formations ranging in age from pre-Cambrian to the present. The formations include metamorphics such as gneiss, quartzite, and schist, as well as sedimentaries, such as conglomerate, shale, limestone, and tuff. The Weber River is the principal stream to enter the east side of the Great Salt Lake. It flows from the Wasatch Mountains in a generally westerly direction across the area and is joined near Ogden by its main tributary, the Ogden River.

In past geologic times, the Weber River and other streams have deposited eroded material to depths of several thousands of feet in the basin of ancient Lake Bonneville and its remnant, the Great Salt Lake. The highest shore line of the once extensive Lake Bonneville was at an elevation of about 5,200 ft, and the present elevation of the Great Salt Lake is about 4,200 ft. The steep front of the Wasatch Mountains shows a number of remnant shorelines at different elevations. Broad alluvial benches and terraces extend westward from the mountains.

Geologically, the significant features of the area are as follows:

1. The Wasatch fault, which is roughly parallel to and just west of the Wasatch Mountains.

2. Little Mountain, a pre-Cambrian block that rises a few hundred feet above the surrounding plain and is located about 15 miles west of Ogden, and near the shore of the Great Salt Lake.

3. The section of lake basin sediments—several thousand feet thick—deposited in the Bonneville-Great Salt Lake Basin. This section contains numerous artesian aquifers.

Several hot springs in the area support the belief that warm mineralized water rises from considerable depth along the (Wasatch) fault zone and is one source of the ground-water contamination, or brackish water, referred to.

Geophysical investigations.—Geophysical investigations were made on the Weber Basin Project during a 6 weeks period in the fall of 1955. The seismic reflection method—widely used in searching for oil—was employed to measure the depth to sand and clay layers and trace them from point to point in the area, and thus provide subsurface information. A set of "shallow" seismic reflection equipment was loaned to the USBR, Dept. of Interior (USGS) by the Geophysics Branch of the United States Geological Survey. That organization also made available the services of an experienced geophysicist to assist in starting the work.

The geophysical investigations were cooperative in another respect. At the time the seismic work was underway, the USGS was conducting gravity meter field studies in the area. In the course of these studies, a reconnaissance gravity meter survey of the area was made in which the USBR was interested. A copy of this map, which supplied a broad picture of the slope and configuration of the basement rock in the basin, was made available to the USBR.

Seismic reflection method.—The seismic reflection method is basically similar to marine "echo sounding," whereby the depth to the ocean floor is determined. To make an echo sounding, the time, in seconds, is measured for a pulse of energy sent from a vibrator in the hull of a ship to travel to the floor of the ocean and back to a vibration detector, which is also in the ship's hull. To convert this reflection time to the depth of the ocean floor, one-half of the measured time is multiplied by the speed with which the vibration travels in sea water.

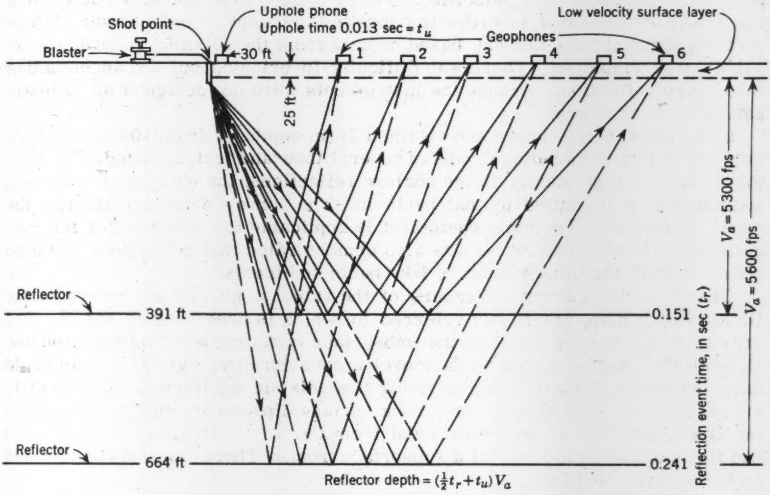
To apply the reflection procedure to map subsurface horizons, such as gravel or clay layers, one-half of the "reflection event" time obtained from the seismic record is multiplied by the speed of seismic waves through the layers involved. The seismic reflection method is illustrated in Fig. 1. On the seismic reflection record, the arrival of reflected seismic wave energy at the geophones is shown by the sharp down-break in trace lines on the seismic record. Time of the reflection event can be scaled to 0.001 of a second.

Seismic field operations.—The first seismic lines were placed near the two deep drill holes put down by the USBR. The seismic records showed numerous reflections from various subsurface horizons. This was significant because it was not known, before the survey was undertaken, if the lake sediments would yield reflections.

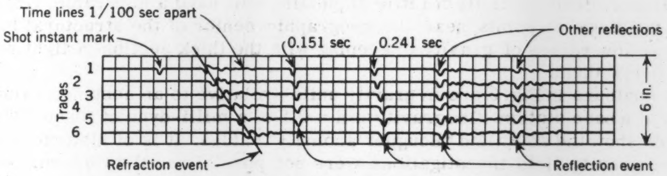
Early in the survey, the speed with which seismic waves travel in the lake beds was measured in three deep drill holes to ascertain the value of wave velocity to use in computing the depths to the different reflecting layers. These velocity measurements were made by lowering a special seismic wave detector, or "geophone," into the drill hole to a measured depth and timing the wave from an explosive charge detonated in a shallow "shot hole" at the surface.

Correlation of reflections and depths to sand and clay layers, shown on drill logs, was studied carefully in the beginning of the investigation because it was a critical point as to the usefulness of the seismic method in this problem. It was found at one drill hole, for example, that twelve reflections were "picked" from the seismic records, which came from depths of 213 ft to 1,130 ft. The total depth of this drill hole was 1,200 ft. These twelve reflections correlated with the logged depths to changes from sand to clay or clay to sand, and so forth. The maximum discrepancy in correlation was 7 ft.

Correlation of reflections.—Study of the reflection seismograms and drill logs showed that a sufficiently close agreement was demonstrated to permit reflections and/or the beds of sand or clay to which they related, to be correlated between seismic stations—or "spreads"—that were from 1/2 to 1-1/2 miles apart.


Uphole phone
Uphole time 0.013 sec = t_u
Shot point
Low velocity surface layer
Geophones
Blaster
30 ft
1
2
3
4
5
6
25 ft
$V_a = 5300$ fps
$V_a = 5600$ fps
Reflector
391 ft
0.151
Reflector
664 ft
0.241
Reflection event time, in sec (t_r)
Reflector depth = $(\frac{1}{2}t_r + t_u)V_a$

(*a*) EXAMPLE OF DEPTH CALCULATION


Time lines 1/100 sec apart
Shot instant mark
0.151 sec
0.241 sec
Other reflections
Traces
1
2
3
4
5
6
6 in.
Refraction event
Reflection event

(*b*) SEISMIC REFLECTION RECORD

FIG. 1.—SEISMIC REFLECTION METHOD

In the course of the seismic survey, seventy-four depth determinations and three sets of drill hole velocity measurements were secured. These depth points were established on seismic spreads located along traverses which followed section line roads across the area. These traverses permitted reflection cross sections to be drawn which were tied to basement rock at the outcrop and/or to deep wells. A total of over 60 miles of exploration traverse was completed. One of these reflection cross sections is shown in Fig. 2. The reflection depths on the cross section are plotted from ground surface and do not necessarily relate to the scale showing depth in feet.

Deep reflections.—On some of the traverses, reflections were observed on the seismic records that came from depths of 4,000 ft to 6,000 ft or more. These depths were greater than the maximum of 3,000 ft to which it had been possible to measure wave velocities directly in deep drill holes. To determine the velocities assumed in order to compute the depths of these lower reflectors, an approximate method, based on data from the seismic records themselves, was employed. There was difficulty in bringing out and accentuating these deep reflections because the instruments were not designed for investigations at such depths.

In the area, reflections were obtained from depths of from 100 ft to 200 ft, or less, up to and including depths of several thousand feet, as noted. Because there was interest mainly in the shallow reflections, the equipment employed was the type most suited to that problem. The survey demonstrated that the "shallow reflection" seismic equipment is applicable to ground-water investigations in artesian basins. It was also demonstrated that reflections could be obtained from the thick section of lake basin sediments.

Gravity meter survey.—Contours of the value of gravity are shown on the USGS gravity map, previously referred to, which is presented as Fig. 3. The gravity meter survey outlined the subsurface basin and the configuration and slope of the basement rock surface over a considerably larger area than could have been economically mapped using the seismic equipment. The gravity survey indicated that the basement rock surface dips toward the lowest part of the basin, not only from the east and from the west, but also from the north and the south along the axis of a subsurface trough. These dips were confirmed by the seismic findings.

The relation of gravity force to the depth of the basement rock is that at any station where a gravity meter is set up the gravity as measured will reflect the average density of the column of material from the surface down. Since pre-Cambrian rocks of the Wasatch Range and of Little Mountain are of a higher density than lake basin sediments, gravity meter readings near the mountains and in the vicinity of Little Mountains will have a high value. Gravity meter readings at points near the geographic center of the structural basin will have low values of gravity in keeping with the thick section of light sediments below them.

The contours in Fig. 3 are in gravity units—relative to an assumed value of gravity at a base station. One gravity unit = 0.1 of a milli-dyne of force. These contours show the so-called "Bouguer anomaly" values. It is of historic interest that gravity field investigations were not possible until an instrument of the necessary accuracy had been developed, which was in about 1900, in Hungary. This instrument was the torsion balance perfected by the physicist, Baron Roland Von Eotvos.

USBR deep test well.—A third test well, previously referred to, was drilled by the USBR to a depth of 3,006 ft using rotary drilling equipment. It was located on the basis of information supplied by the geophysical surveys.

As part of the drilling contract, an electrical log of the hole was specified which included; (1) measurements of natural electrical current—"Spontaneous Potential," shown on the resulting electrical log as the "S. P. Curve," (2) measurements of the electrical resistivity of formations using various spac-

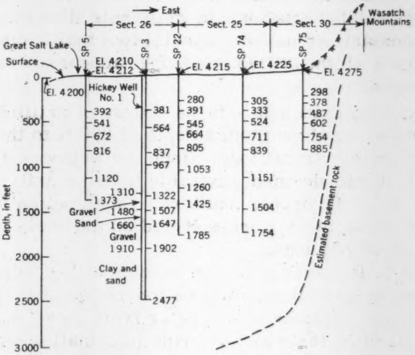

East
Sect. 26
Sect. 25
Sect. 30
Wasatch Mountains
Great Salt Lake
Surface
SP 5
SP 3
SP 22
SP 74
SP 75
El. 4210
El. 4212
El. 4215
El. 4225
El. 4275
El. 4200
Hickey Well No. 1
0
500
1000
1500
2000
2500
3000
Depth, in feet
392
541
672
816
1120
1373
381
564
837
967
1310
1322
1480
1507
1660
1647
1910
1902
2477
Gravel
Sand
Gravel
Clay and sand
280
391
545
664
805
1053
1260
1425
1785
305
333
524
711
839
1151
1504
1754
298
378
487
602
754
885
Estimated basement rock

FIG. 2.—REFLECTION CROSS SECTION VV'

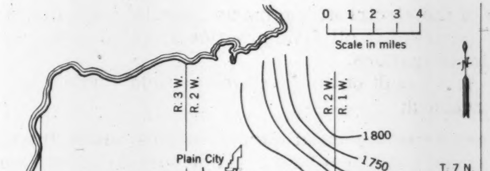

0 1 2 3 4
Scale in miles
R. 3 W.
R. 2 W.
R. 2 W.
R. 1 W.
R. 4 W.
R. 3 W.
1800
1750
1700
1650
1600
1550
1600
1650
1700
1750
1800
1850
1900
1950
Plain City
T. 7 N.
T. 6 N.
Little Mountain
River
Ogden
River
Ogden
Weber
T. 6 N.
T. 5 N.
Weber
River
T. 5 N.
T. 4 N.
GREAT SALT LAKE
Kaysville
T. 4 N.
T. 3 N.

FIG. 3.—GRAVITY CONTOURS IN THE EAST SHORE AREA

ings of the three contact electrodes that were sent down the drill hole, and (3) a "caliper log" by which variations in drill hole diameter are measured by spring-actuated contact points on a special tool that traverses the drill hole. In bore hole logging all quantities measured appear as separate lines on the graphic logs obtained.

Special tests in deep drill hole.—In the course of drilling the deep test well, a number of potential aquifers from a depth of 900 ft to the bottom of the hole were indicated by the electrical logs. Since the well was drilled by the rotary method, as noted, it was deemed advisable to make drill stem tests on zones identified as the most favorable aquifers. Depth points for setting the sidewall packers for these tests were selected on the basis of interpretations of the caliper and resistivity logs.

In addition, the S. P. curve from the electrical log indicated that the water in the aquifers below 1,400 ft was of questionable quality. The drill stem tests enabled us to secure samples of the water from these aquifers for chemical analysis. The drill stem tests also provide information on, (1) the static fluid pressure in the aquifer, (2) reduction in pressure with time after discharge is stopped. Approximate values of porosity in percent and permeability in millidarcies were supplied by the logging operators, on request, for different sections of the drill hole. These data were determined from the operators' analysis of the electrical logs.

The results of the electrical logging and special tests illustrate that methods commonly employed in oil field practice are also useful and applicable in ground-water investigations.

Summary.—As a result of the geophysical field studies, the following conclusions were reached:

1. Reflections from layers at various depths within the 1,220 ft of sediments drilled at one USBR test hole could be correlated with sand and/or clay layers shown on the drill log. Numerous reflections from below 1,220 ft and to 4,120 ft suggested that there were 2,900 ft of sediments composed of sand, gravel, or clay layers which were not tested at that location.
2. Reflections within the 977-ft section drilled at another location were correlated with logged and known changes of material. Numerous reflections between depths of 977 ft and 5,310 ft suggested that about 4,300 ft of section that likewise might contain sand, gravel, or clay layers were present at that location.
3. Reflection cross sections on which the "estimated basement rock line" was shown, indicated that the basement rock surface dips east from Little Mountain, located west of Ogden, and dips west from the Wasatch Mountains. Outcrops of basement rock at these two localities established a subsurface basin or deep trough whose axis runs roughly north-south through the major portion of the East Shore Area. It was concluded from the reflection data that the basement rock might be as much as 6,000 ft deep in the bottom of the trough, which lies some 5 miles west of Ogden.
4. The gravity meter survey of the East Shore Area outlined the artesian basin and showed the configuration of the basement rock surface in some detail. The gravity meter data effectively supplemented the information secured by the reflection seismic survey as to the position of the lowest portion of the basin and the direction of its axis.

The joint use of these two geophysical methods was shown to be advantageous in ground-water investigations of such artesian basins.

5. The electrical and caliper logging of the deep test hole indicated that several aquifers suitable for further testing were present at certain depths. The logging also suggested the quality of the water contained in these aquifers and supplemented the data obtained from the drilling. Subsequent drill stem tests supplied detailed information on the aquifers selected for study.

Resistivity Field Work in South Dakota.—Resistivity field investigations were carried on almost continuously, from the fall of 1951, through 1953, as part of a ground-water and drainage investigation in the vicinity of the city of Huron, South Dakota. The work was on the Oahe Unit of the Missouri River Basin Project of the USBR.

The region, which was studied by geological mapping, drilling and resistivity depth measurements, lies between Huron, in the east central part of South Dakota, and the city of Redfield, some 36 miles to the north. The area investigated was some 30 miles wide and comprised approximately 1,100 sq miles. A contiguous area north and west of Redfield, containing some 900 sq miles, was investigated somewhat less extensively.

The entire region was covered at various times, in past geological ages, by continental glaciers. When the glaciers retreated for the last time, they left a mantle of glacial drift which varied from a few feet in thickness to a maximum of 300 ft. The Pierre shale, of Cretaceous age, lay beneath the drift at an average depth of 90 ft, according to drill tests.

The drift, from the surface down, consisted of the following; a) an average of some 50 ft of glacial till which was mainly clay, b) a variable thickness of sand or predominantly sandy material which ranged from 7 ft to 155 ft, and c) clay, till, silt, or silty sand, above the shale. The sand bodies carried water and might be essentially continuous over areas of several sq miles. In parts of the area, the sand zones might be lenticular and also separated by layers of till. In yet other localities, the sand might be absent.

The most notable character of the glacial drift was its heterogeneity, both laterally and vertically. These conditions made exploration by drilling difficult and costly. Surface geological mapping was not particularly diagnostic of subsurface possibilities. The resistivity field work was used to supplement core drilling and as a guide in outlining the sand aquifer areas. This was one of the main objectives of the investigation.

The resistivity field crew consisted of four men who worked under the direction of a geologist of the Missouri-Oahe Projects Office. During a 9-month period, prior to August, 1953, they completed over 700 resistivity depth measurements, and, in total, over 1,200 such measurements were made. These measurements were closely tied to the exploration drilling to establish and check criteria for the interpretation of the resistivity depth curves. This procedure was continued during the entire investigation.

It was found that the resistivity depth curves would indicate: a) whether sand was present at a test locality and if so, at approximately what depths. Fig. 4(a) is an example of this condition. The curve of Fig. 4(a) shows the relatively high resistivity of sand; b) whether no sand was present and whether the section was all till above the shale, see Fig. 4(b). The curve of Fig. 4(b) shows the typically low resistivity of glacial till; c) within limits, whether the sand was "clean" as shown by relatively high resistivity; d) also within limits, whether the sand was "dirty" and contained appreciable amounts of silt or clay as shown by a relatively low resistivity; and e) quite unmistakably if the water table were below the top of the sand body as shown by Fig. 5. The curve of

Fig. 5 shows very high resistivity due to dry sand above the water table. Note the small scale of Fig. 6 in comparison with that of Figs. 4 and 5.

The resistivity measurements were not considered as a substitute for, but as a supplement to, drilling. A careful study of the first five hundred resistivity depth curves and the logs of the sixty-six drill holes that provided specific check points, showed that in fifty-three cases, or over 88%, the resistivity measurements gave results that definitely could be correlated with the drill logs. For example, where sand was indicated by the resistivity curve, sand was logged from the drilling and at appropriate depths. Where the curve indicated the section was all till, clay, or silt, the drill indicated such material was present.

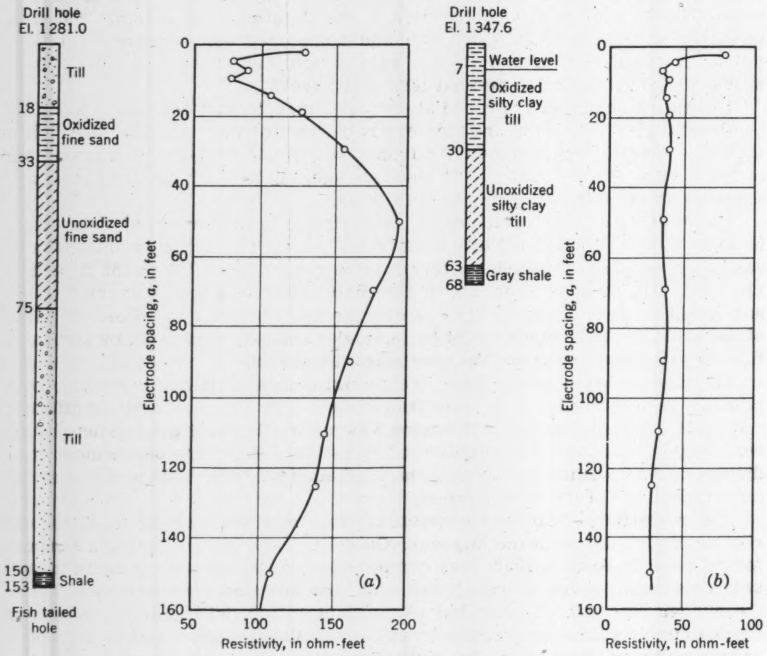


FIG. 4.—RESISTIVITY DEPTH CURVES

In thirteen cases, or some 12%, the correlation was only fair. The resistivity depth curve indications were not "clean-cut" as between definitely sand or definitely clay. These were borderline cases. However, they indicated clearly that drilling was required to learn the true subsurface conditions at these test points.

It was also possible to determine the depth to the shale by resistivity depth measurements. These depths were not required in most cases and depth curves were, therefore, not interpreted for shale depth.

A NEW DEVELOPMENT FOR DETECTING LEAKAGE IN CANALS

The geophysical procedures and their application to ground-water investigations that have been described have followed lines established in oil exploration or the search for ore bodies. A new development is the use of electrical logging in canals to detect leakage.

Field trials of electrical logging to detect probable leaking sections of operating canals were made on the Central Valley Project, California, in October, 1958. The procedure employed was an adaptation of the drill hole electrical logging which has been discussed.

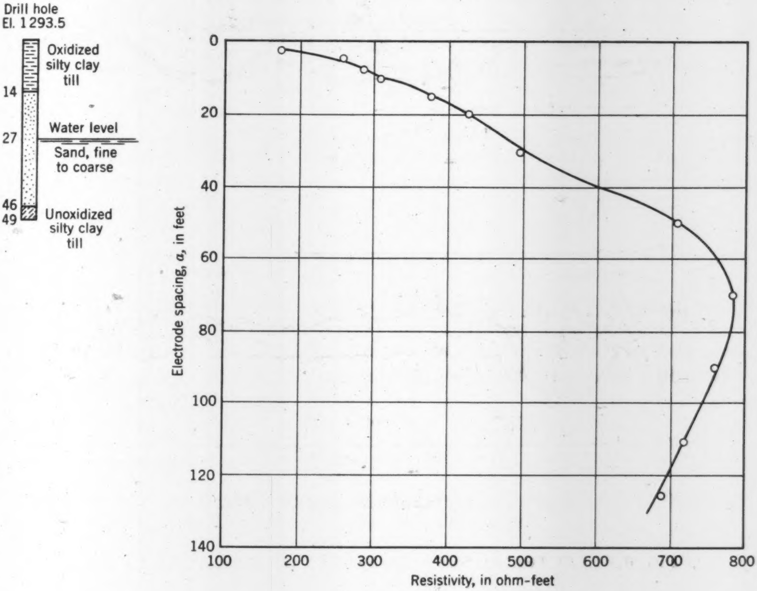
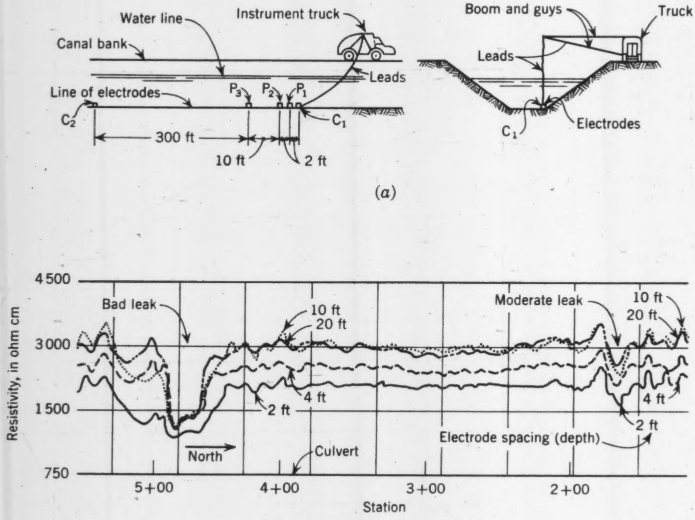


FIG. 5.—RESISTIVITY DEPTH CURVE 32

During the trials, some 7 miles were logged on selected reaches of four major canals that were well suited to such testing. The work was done under the USBR's Lower-Cost Canal Lining Program by a contractor who has specialized in geological and geophysical investigations. How canal logging is done is illustrated in Fig. 6, in which P is voltage electrode and C is current electrode.

Electrical logging permits the continuous measurement and recording of the electrical resistance (resistivity) of materials comprising the bottom and/or banks of a canal. This is similar to the electrical logging of a drill hole where variations in the materials are shown by differences in measured resistivity.

To log a water-filled canal, contact was made with the ground by four cylindrical lead electrodes which were placed in a line at measured distances apart and laid on the bottom of the canal. The electrodes were connected by rubber-covered wire lines leading to a current source and to recording equipment in an instrument truck. An electrical current was sent through the ground between the outer pair of electrodes and the voltage between the inner pair was measured as the truck moved along the canal bank dragging the string of electrodes behind it. The electrodes indicated in Fig. 6 were dragged along the west band of the canal just below the surface of the water. Variations in resistivity were shown as variations in measured voltage by the moving pens on the recorder chart, which unrolled as the instrument truck moved forward.


Water line
Instrument truck
Canal bank
Leads
Line of electrodes
P_3
P_2
P_1
C_2
C_1
300 ft
10 ft
2 ft
Boom and guys
Truck
Leads
C_1
Electrodes
4500
3000
1500
750
Resistivity, in ohm cm
Bad leak
10 ft
20 ft
Moderate leak
10 ft
20 ft
4 ft
2 ft
4 ft
2 ft
North
Culvert
Electrode spacing (depth)
5+00
4+00
3+00
2+00
Station

(*a*)

(*b*) ELECTRICAL LOG OF CANAL

FIG. 6.—ELECTRICAL SEEPAGE SURVEY OF IRRIGATION CANAL

The spacing of the electrodes controlled the depth to which the resistivity of the formations were measured. Logging was done, simultaneously, at a 2-ft spacing and a 10-ft spacing since the recorder had two pens. A separate traverse was required to log at 4-ft and 20-ft spacings, after changing the electrodes.

Drill hole logging shows that different formations and their character can be identified on electrical logs. In like manner, seepage locations along a canal may be identified if they are markedly different in resistivity from adjacent sections. At present, because of limited experience, it is necessary to correlate very carefully the measured resistivity in a canal section under study with information obtained by drilling.

For example, water lowers the resistivity of earth materials. Therefore, local zones along a canal that show relatively low resistivity, at all depths logged, might indicate seepage. Interpretation must be based on a knowledge of geological conditions and of the material in which the canal is constructed. This is because wet clay, which may be water tight, also shows a low resistivity. In contrast to this, sand and gravel, which show a high resistivity in general, may likewise represent a leaky section.

Results of canal logging tests, and as that previously noted suggests, are on a preliminary basis. It is felt, however, by those engineers most familiar with it, that the electrical logging of canals to detect seepage has promise of being effective to that end.

If it is found that the electrical logging of canals will indicate leaky zones, it will be very advantageous in the operation and maintenance of canal systems. Main reliance is placed on ponding tests to detect leakage and these are quite expensive and require taking the canal out of service during the test.

CONCLUSIONS

Geophysical procedures have been developed largely in the petroleum field. This paper has pointed out that under certain conditions these procedures are also applicable in the location and utilization of ground water. The examples presented herein were selected from among twenty-five or thirty with which the writers are familiar.

ACKNOWLEDGMENTS

The authors are grateful to Don R. Mabey, geophysicist, and to the Director of the USGS for the map indicated as Fig. 3 in this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3138

INSTALLATION OF DRAIN TILE FOR SUBSURFACE DRAINAGE

By John G. Sutton,[1] F. ASCE

SYNOPSIS

This paper reports current progress in installation of tile drains for subsurface drainage. The Soil Conservation Service provided technical assistance on installation of 24,059 miles of tile drains during the 1959 fiscal year. The paper gives benefits of tile drainage, and describes tile used in combination with surface drainage and current procedures used by the Soil Conservation Service in making drainage investigations. Recent changes in the American Society of Testing Materials specifications for clay and concrete drain tile are discussed. Recommendations are given for use of concrete tile under acid and alkali conditions. Recommendations for gravel filter requirements are given.

INTRODUCTION

In a prior paper,[2] the author discussed drainage in the humid areas of the United States. The earlier paper described extent of drainage, some historical developments, drainage enterprises, programs of Federal and state governments which provide assistance, and some of the engineering problems.

Tile drainage systems which are designed to provide drainage for agricultural lands are the subject of this paper. Herein, it is planned to cover in more detail several phases of the practice of tile drainage where substantial progress has been made during the last few years, particularly where current practices have adapted research findings and otherwise differ from published

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Note.—Published essentially as printed here, in September, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2591. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Drainage Engr., Soil Conservation Service, U.S. Dept. of Agric., Washington, D. C.

[2] "Drainage in the Humid Areas of the United States," by John G. Sutton, Transactions, ASCE, Vol. 122, 1957, p. 115.

material generally available. Drainage systems for airports, urban areas and special installations are not discussed.

The Soil Conservation Service in the U. S. Department of Agriculture provided technical assistance on the installation of 24,059 miles of tile drains during the fiscal year 1959. Tile drainage is an essential practice in carrying out a conservation program, because it is necessary for the efficient use of a substantial portion of productive flat lands which are least susceptible to erosion.

In the humid areas, tile for agricultural lands usually consist of either clay or concrete drain pipe, usually 1 ft long, laid in trenches generally 30 in. to 54 in. deep. Nearly all the farm-drain laterals are 4 in. to 6 in. in inside diameter. Mains up to 24-in. diameter, and occasionally larger sizes, are used. In laying drain pipe a small gap is left between the ends of the tile. These gaps permit water to enter the tile. In irrigated areas, tile lines are ordinarily 5 1/2 ft to 8 ft deep. Bell-end and tongue and groove pipe are used extensively in drainage of irrigated lands.

Bituminized fiber pipe is used in many installations in Northeastern states and less frequently in other areas. Plastic pipe has been used in limited amounts. Covered drains of wood box, metal pipe, and other materials are occasionally used.

Tile drains remove gravity water from the soil, stabilize the groundwater table, and collect seepage. Underdrainage is used most frequently in humid areas for imperfectly drained soils subject to a high water table. Such soils need to be permeable enough to permit economical spacing of drains.

In irrigated areas high water conditions are due in most cases to seepage from reservoirs, canals, laterals, and irrigated lands. Saline and alkali salts rise to the surface in many of these areas resulting in extensive damages to crops and land. Most tile installations in western irrigated areas are needed for reclamation or continued use of such lands.

In all cases the values of the crops to be grown must be increased sufficiently to return net profits which can pay off the high investment required for tile drainage. Some soils are permeable enough so that tile drainage is not needed. Other soils may be drained, even though less effectively, by surface drainage with the result that tile drainage is not justified. A thorough knowledge of soils and reactions of soils and crops after tile drains are installed is essential.

BENEFITS AND EFFECTS OF TILE DRAINAGE

The tile drainage of many wet soils results in improved crop yields and more profitable farming. The benefits discussed subsequently are obtained through drainage in conjunction with necessary soil amendments and soil conservation crop rotations.

Soil aeration is necessary for favorable bacterial action and in order to permit rapid diffusion of the soil air to the atmosphere and vice versa. Tile drains aid these processes by lowering the groundwater usually as much as 12 in. to 18 in. within 24 hr after a heavy rain, thereby providing a zone of aerated soil soon after saturation occurs. Tile drains stabilize the normal groundwater at a lower level than in an undrained soil and provide a deeper root zone on the average.

Freezing, thawing, and droughts influence the development of the soil structure and soil permeability following installation of tile systems. In a well-drained soil, the gravity water moves downward and outward through tile lines instead of saturating the soil and being removed slowly by evaporation and transpiration. The capillary moisture which is utilized by roots of plants is not removed by tile drains.

Tile drainage of wet soils encourages conservation farming, which generally includes the use of grasses and deep-rooted legume crops such as sweetclover and alfalfa. These crops do not thrive in wet soils. The roots of grasses and legumes open up pore spaces in drained soils and encourage formation of soil aggregates. Thus, after tile drainage, the permeability of tight soils improves with conservation farming and good crop rotations.

Well drained soils warm up sooner and can be cultivated earlier in the spring than wet soils. This benefit of tile drainage is especially important in northern climates. The date of seed germination is higher and a good stand of the crop is obtained on the drained soil.

A deep root zone encourages early development of deep-rooted and hardy plants. On well-drained fields, crops are better able to withstand drought due to the deep root system. Soil amendments and fertilizers are utilized effectively by crops grown on a well-drained soil.

The fields can be cultivated with less delay after rains and tractor cultivation is more efficient because the soil dries uniformly and it is not necessary to cultivate around wet spots or parts of fields. Equipment is less likely to mire down. Improved efficiency in cultivation is one of the most important benefits of tiling.

In arid areas widespread salt damage occurs, due to the deposition of salts near the surface from capillary water rising through the soil. Deep drains lower the water table and result in a downward movement of salts in the soil. This lowers the salt concentration in the surface layers of soil and improves crop growing conditions.

The improvement of public health conditions through proper drainage is particularly important in many locations. Tile drainage has an advantage over drainage by open ditches through elimination of standing water where mosquitoes may breed.

SURFACE DRAINAGE NEEDED WITH TILE SYSTEMS

The best drainage is most frequently obtained by a combination of open and closed drains. It is usually most economical to remove surface water from tiled fields by installing shallow surface ditches constructed to grade which do not interfere with cultivation. Land grading and smoothing which eliminate shallow depressions also is beneficial. The surface and subsurface drainage systems are usually complementary on the same area and need to be planned together.

On some soils there is a choice as to whether tile lines or open drains should be used to provide subsurface drainage. Tile drains save land and result in cheaper cultivation. Open ditches spaced closely enough to control the water cut up the field into relatively narrow strips and use much land. Open ditches may harbor weeds and are more expensive to maintain. On the other hand, surface drainage systems are cheaper in first cost and are the only practical solution for some soils. In some areas farming may be carried out initially by improving the surface drainage. Later, it pays to install tile drainage.

Research is needed to determine the increment by which yields of various crops are increased due to tile drainage. One example was an experiment in Ohio in 1958 where corn yielded 120 bushels per acre on a tile drained field with surface drainage and 80 bushels per acre on a field not tiled but having a comparable surface drainage system.

Research also is needed to determine the effective use of land grading and smoothing for drainage used in conjunction with tile drainage. Land grading for drainage in recent years has proven very beneficial. Data are needed to determine areas where land grading and tile should be used together and how specifications for the two practices should be modified when used together.

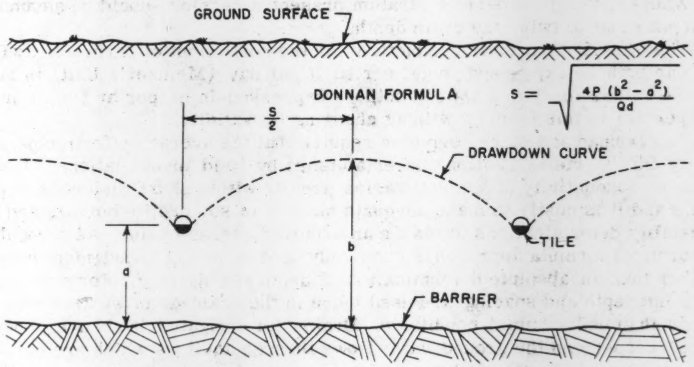

FIG. 1.—TILE SPACING FORMULA

THEORIES OF FLOW OF WATER TO TILE DRAIN

Several theories covering the flow of water to tile drains have been proposed in recent years. Attention is called to the excellent discussion of this subject by J. V. Schilfgaarde, D. Kirkham, and R. K. Frevert.[3] These theories have modified the approach of the drainage engineer towards the solution of many tile drainage problems. Considerable progress has been made towards the rational design of drainage systems using such theories. However, there are still many problems to be solved before a fully rational approach to drainage design is established, including field methods of determining the effective soil permeability.

Among these formulas is the one commonly known as the Donnan Formula,[4] which is typical. The formula was developed for relief drains and is based upon certain barrier conditions. This is illustrated in Fig. 1. The Donnan Formula may be expressed as follows:

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[3] "Physical and Mathematical Theories of Tile and Ditch Drainage and Their Usefulness in Design," by Jan van Schilfgaarde, Don Kirkham, and R. K. Frevert, Agric., Sta., Iowa State College, Research Bulletin No. 436, February, 1956.

[4] "Drainage Investigations in Imperial Valley," by W. W. Donnan, G. B. Bradshaw, and H. F. Blaney, (10-yr summary), U. S. Dept. of Agric., Soil Conservation Service, SCS-TP-120, 1954.

$$S = \sqrt{\frac{4P\left(b^2 - a^2\right)}{Q_d}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which S = spacing of the tile lines in feet; P = hydraulic conductivity or coefficient of permeability expressed in gal per sq ft per day (Meinzer's Unit); or, cu in. per sq in. per hr usually abbreviated in. per hr; b = distance from the average tile depth to barrier stratum at the midpoint between the tile lines, in feet; a = distance from the average tile depth to barrier stratum, in feet; and Q_d = quantity of water to be drained expressed as gal per sq ft per day (Meinzer's Unit) or in cu in. per sq in. per hr, usually abbreviated, in. per hr.

Where there is no barrier stratum present, a barrier should be assumed at a depth equal to twice the drain depth.

The units for hydraulic conductivity or the coefficient of permeability P and Q_d can both be expressed in gal per sq ft per day (Meinzer's Unit) in Eq. 1; or the values for P and Q_d can both be expressed in in. per hr (cu in. per sq in. per hr) in this formula without changing its validity.

The Donnan and other formulas require that the average effective permeability and barrier conditions be established by field investigations. The hydraulic conductivity of the soil varies greatly within short distances in many soils and it is costly to make adequate number of soil profile borings and permeability determinations to insure an accurate average value. As a result the theoretical formula approach is commonly used as an aid to judgment in design rather than an absolute determination of depth and spacing. Formulas to determine depth and spacing are used more in the drainage of western irrigated lands than under humid conditions. Drains for irrigated lands are generally much deeper and the ground water flows through aquifers which are not influenced greatly by roots of plants or freezing and thawing. Hence, the conditions assumed in developing ground flow formulas are more nearly present under such conditions.

EFFECTS ON CROPS AND SOILS AS AN INDEX OF TILE DRAINAGE REQUIREMENTS

In the humid area the permeability values (hydraulic conductivity) of soils above tile depths vary greatly from season to season depending upon such factors as frost action, quantity and depth of roots and drying and cracking of soils and soil aggregations. The soil structure under such conditions varies greatly and results in variations in permeability. Under such conditions the spacing of drains is generally established by what may be termed drainage field trial studies. In Midwestern areas the effects of drains on crops of corn are observed. When drains are spaced too far apart there is a noticeable lag in plant growth in the center area between two tile drains. The tiles are therefore spaced more closely together on the next installation until a uniform cropping pattern can be obtained throughout the field. Crop yields for various depths and spacing of tile have been computed.

When drains are too far apart the fields dry unevenly after rains and areas away from the tile remain wet too long. These wet areas are readily identified by the farmer in cultivating his crops. Such observations also aid in establishing proper depth and spacing criteria. On many soils the effects of tile on draw down of the water table are observed by installing ground water observation wells. The combined results of these kinds of observations have enabled the establishment of recommendations for depth and spacing which generally produce good results.

SOIL CONSERVATION SERVICE TECHNICAL HANDBOOKS AND DRAINAGE GUIDES

One of the principal activities of drainage engineers in SCS is to develop standards, criteria and handbook material. During the last two years the SCS has issued several chapters of the National Engineering Handbook covering Agricultural Drainage Subjects. These drainage chapters are for in-Service use but may be consulted at SCS field offices. Included among these chapters is one covering open ditches which includes design procedures for open drainage ditches. Also, chapters on drainage surveys, investigations and reports and subsurface drainage of western irrigated lands have been issued. The chapters covering tile drainage in the humid area and tile systems and appurtenances are also available. After these chapters have been in use for a few years, it is planned to make them available to the public through the Government Printing Office. The state offices of SCS have issued state engineering handbooks for work unit staffs. They include chapters on drainage and provide criteria for planning most drainage systems.

Most states having extensive drainage activities have completed or are preparing drainage guides. Drainage guides represent a fairly recent concept in compiling and making available information on drainage design standards and criteria applicable to a specific area. National and state handbooks cover general procedures applicable to a wide variety of drainage conditions and installations.

Drainage guides set forth the detailed local design requirements for drainage systems within certain geographical areas, each with its particular combinations of climate, soils, and land use. These guides represent the best accumulated record of drainage experience of farmers, ranchers, and technicians. They cover such requirements as water table control, drainage coefficients, depth and spacing of drains, and requirements for investigation, layout, and construction. Drainage guides constitute the Soil Conservation Service standards of drainage engineering in the local areas. They are established within the general criteria in the National and State Engineering Handbooks and other standards.

INVESTIGATIONS FOR TILE SYSTEMS IN HUMID AREAS

In the humid area, investigations for tile drainage systems may consist only of an on-site examination or such investigations may require extensive surveys, soil borings, and water table studies. Under many conditions, the experienced technician may install complex systems with a minimum of surveys and investigations. The experienced drainage engineer will obtain all the necessary facts but will make instrument surveys only where required. His procedure is likely to follow the following outline:

I. Need and Feasibility of Installing a Tile Drainage System
  A. Indications of high water table, or seepage
    1. Soils examination
    2. Soil auger borings
    3. Plant indicators
    4. Crop yields
    5. Hazards of cultivation
    6. Farmer experience
  B. Will increases in crop yields and other benefits increase returns sufficiently to pay costs?
    1. Crop yields before and after tiling
    2. Comparisons with similar soil, topography, and crop situations
    3. Other farm benefits
    4. Approximate costs of necessary drainage systems
II. Planning Drainage System
  A. Adequacy of outlet
  B. Determine fall available, using engineer's level
  C. Location of mains, depth and size (usually requires a profile)
  D. Depth, spacing, and size of laterals
  E. Tile lines to intercept seepage (based on soil borings)
  F. Structures and special construction items
  G. Special requirements due to acid, alkali, depth, grade, sand, or rock
  H. Cost of system.

INVESTIGATIONS OF WESTERN IRRIGATED AREAS

In the western irrigated area the subsurface drainage investigations are more intensive. In addition to the general type investigations previously listed, the drainage investigations for irrigated lands generally include the following: (a) Topography; (b) Soil-profile investigations; (c) Hydraulic conductivity measurements; (d) Ground-water investigations; (e) Observation wells; and (f) Piezometers. It is generally necessary to obtain or prepare a topographic map of all or part of the area to be drained.

The most detailed drainage investigations are required in the western irrigated regions where saline and alkali salts are present. The drainage engineer needs to recognize the salt problem and ordinarily he requests assistance of a soils technician to study the salt concentrations and chemical amendments which are needed. The reclamation of saline and alkali soils is outside the scope of this paper. However, the installation of subsurface drainage is usually the first step in reclamation of such salty soils where reclamation is feasible. Generally, the reclamation may require one or more of the following:

1. Drainage
2. Soil amendments such as gypsum
3. Leaching by impounding water on the area
4. Selection of adapted crops

Excellent information on the reclamation of saline and alkali soils is available in the handbook on the subject prepared by the U. S. Salinity Laboratory.[5]

SOIL-PROFILE INVESTIGATIONS

In subsurface drainage work, the texture and permeability of subsurface materials need to be determined generally to depths of 10 ft to 20 ft or more, particularly in deep alluvial sediments and where artesian pressures may exist. The success or failure of a subsurface drain depends on how well it has been located with regard to permeable layers. Exploratory borings must be made to supplement the usual conservation soil-survey information so as to obtain more detailed data especially below the 5-ft depth.

The location and spacing of exploratory subsurface borings must be based largely on a knowledge of soils, local geology, and experience gained in a particular work area. Borings must be spaced close enough to permit the location and correlation of subsurface strata as they pertain to drainage. In alluvium where sediments are heterogeneous materials deposited in a complex pattern, the spacing of borings may be 200 ft or less, and in homogeneous residual soils, a spacing of 500 ft to 1,000 ft or more may be satisfactory. In practical field application the usual procedure is to select a grid spacing based on previous experience; start subsurface explorations and attempt to correlate the data from the various borings progressively. If the correlation between borings is poor or lacking, the original grid spacing may need to be reduced and the correlation process repeated. It often develops that in certain parts of an area correlation may be good, whereas in other sections, it may be poor. In this latter case the grid pattern should be supplemented with additional borings to the extent that the subsoil and substrata configuration is made clear.

Borings on large areas should be located on a grid pattern, usually rectangular. The lines of borings may be oriented in any convenient direction to fit the area, but it is preferable to have one axis of the grid orientated fairly closely to the general direction of groundwater movement. When the spacing and grid pattern have been established, guide stakes can be placed at field boundaries and interior stakes can be sighted in. If a topographic survey is to be made, the locations of borings can be tied in on this survey.

The depth of borings should be at least 1-1/2 times the drain depth. Under average conditions in irrigated areas a 10-ft depth is considered adequate. A few deeper borings, 15 ft to 20 ft, interspersed with the others, should be made to determine the composition of underlying strata.

Fig. 2 shows a chart for recording soil borings for drainage investigations. It is especially important to determine the depth and thickness of the most permeable aquifers in order that drains may be installed at the most effective depth. On this chart are recorded the estimates of permeability which are made in the field. Even though such estimates may not have a high degree of accuracy, they indicate the relative permeability of various soil layers which is of primary importance in planning drains.

HYDRAULIC CONDUCTIVITY

The terms "hydraulic conductivity," "permeability," and "coefficient of permeability," although having different technical meanings, are quite often used

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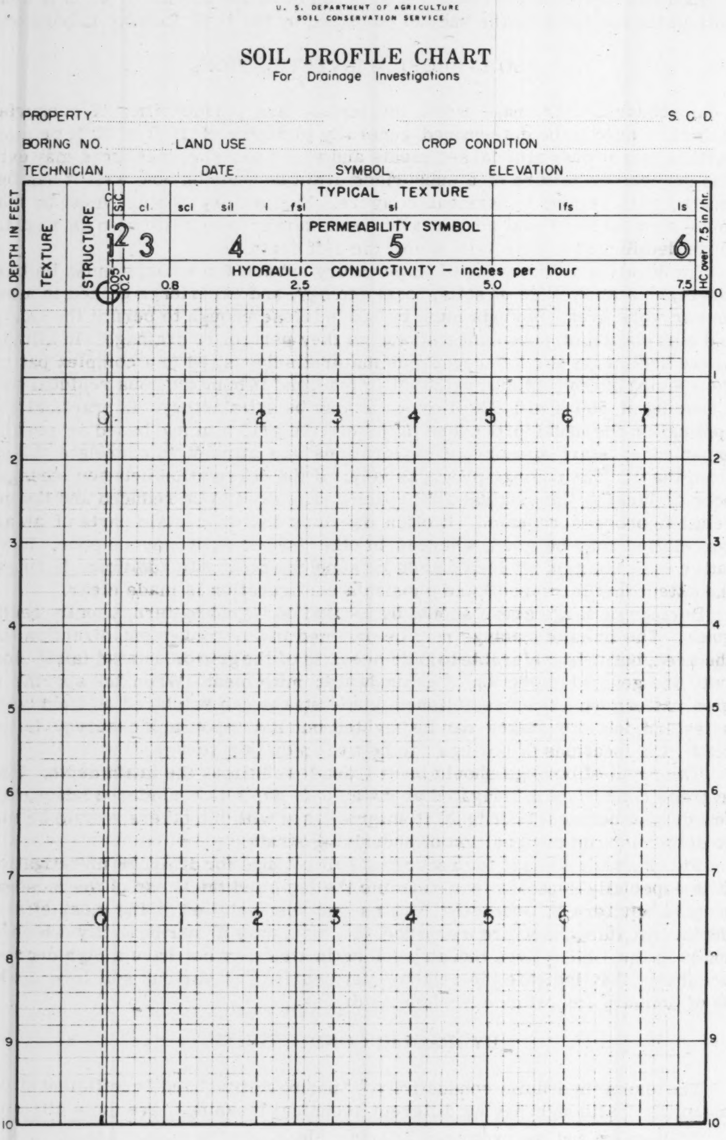
[5] "Diagnosis and Improvement of Saline and Alkali Soils," by Salinity Lab. Staff, Agric. Handbook No. 60. U. S. Govt. Printing Office.



U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL PROFILE CHART
For Drainage Investigations

PROPERTY .... S. C. D.
BORING NO. .... LAND USE .... CROP CONDITION ....
TECHNICIAN .... DATE .... SYMBOL .... ELEVATION ....

0 DEPTH IN FEET
TEXTURE
STRUCTURE
TYPICAL TEXTURE
c
sic
cl
scl
sil
l
fsl
sl
lfs
ls
PERMEABILITY SYMBOL
1
2
3
4
5
6
HYDRAULIC CONDUCTIVITY – inches per hour
.005
0.2
0.8
2.5
5.0
7.5
HC over 7.5 in./hr.
0
1
2
3
4
5
6
7
8
9
10
0
2
3
4
5
6
7
0
2
3
4
5
6

FIG. 2.—SOIL PROFILE CHART

INSTRUCTIONS

1. Make notations of texture, structure in columns at left. Note other conditions, such as wetness, sand stringers, on chart opposite depth.
2. Estimate hydraulic conductivity of each layer, considering texture, structure, pH. See instructions below.
3. Plot permeagraph on chart
4. If hydraulic conductivity exceeds 7.5 in./hr., record at right margin.

TEXTURE LEGEND		STRUCTURE LEGEND
Clay ------------ c	Loam ------------ l	Massive ------- m
Silty clay -------- sic	Fine sandy loam --- fsl	Platy ---------- pl
Sandy clay ------ sc	Sandy loam ------- sl	Prismatic ------ pr
Silty clay loam --- sicl	Loamy fine sand -- lfs	Blocky --------- bk
Clay loam -------- cl	Loamy sand ------- ls	Granular ------- gr
Sandy clay loam -- scl	Sand ------------- s	Single grain ---- sg
Silt -------------- si	Coarse sand ---- cos	
Silt loam --------- sil	Gravel ----------- gr	

ADJUSTMENTS IN HYDRAULIC CONDUCTIVITY ESTIMATES FOR INFLUENCE OF STRUCTURE AND pH

It is generally accepted practice to classify soils with respect to texture and then to adjust the hydraulic conductivity, "HC" estimates with respect to soil structure, alkali and other influencing factors. Soil texture refers to the relative proportions of the various size groups of individual soil grains in a mass of soil. Structure refers to the condition of the soil grains, (clay, silt, sand, etc.), in the way they are arranged and bound together into aggregates with definite shape.

Aggregate length and thickness has an effect upon the "HC". The overlap of aggregates having a horiz. axes 3 or 4 times longer than the vertical can have a marked effect upon "HC". The "HC" of stratum can be changed by cracks, crevices and fractures. Some fractured, (block structured), clay or shale stratum are often much higher than sands or gravel stratum. The "HC" of sandy stratum are generally higher when the grains are round and about the same size than when they are irregular in shape and of different sizes. Flat grains tend to overlap and reduce "HC" rates. The matrix rather than the coarser material in a gravel or cobble stratum govern the "HC" and should be used as a basis in estimating the "HC".

The following two tables are suggested as a possible guide in rating the "HC" of various stratum during a drainage soil survey. It is not anticipated that these tables will replace laboratory analysis and actual field "HC" measurements but that they will be used as estimates and bolstered or revised as additional experience and data becomes available.

PERMEABILITY SYMBOL RANGE for TEXTURE-STRUCTURE CORRELATION

STRUCTURE / TEXTURE	MASSIVE	PLATY	PRISMATIC	BLOCKY	GRANULAR	SINGLE GRAIN
	PERMEABILITY SYMBOL RANGE					
CLAY	1	1 - 3	1 - 2	1 - 6	2 - 5	
SILTY CLAY LOAM	2	1 - 3	1 - 3	2 - 5	2 - 5	
CLAY LOAM	2 - 3	2 - 3	2 - 3	2 - 5	3 - 5	
SILT LOAM	3	3	2 - 3	2 - 4	3 - 4	
LOAM	3 - 4	2 - 4	3 - 4	4 - 5	4 - 5	
SANDY LOAM		3 - 4	4	4 - 5	4 - 6	4
LOAMY FINE SAND				4 - 5	5 - 6	5
SAND						6
COARSE SAND						7

ESTIMATE of ALKALI INFLUENCE on HYDRAULIC CONDUCTIVITY by pH COLOR INDICATORS

SOIL TEXTURE	pH ESTIMATES FROM COLOR INDICATORS			
	8.6	9.0	9.6	10.0
	HC REDUCED (%)	HC REDUCED (%)	HC REDUCED (%)	HC REDUCED (%)
CLAY	20	40	70	90
SILTY CLAY LOAM	15	30	60	80
CLAY LOAM	15	25	50	70
SILT LOAM	15	25	50	70
LOAM	10	20	40	50
SANDY LOAM	5	15	35	45
LOAMY FINE SAND	5	15	30	40
SAND	5	10	25	40

FOR DRAINAGE INVESTIGATIONS

in the same sense. The "coefficient of permeability" has been defined[6] as the rate of flow of water through a unit cross-sectional area under a unit head during a unit period of time. The term "hydraulic conductivity" is defined[7] as the coefficient k in Darcy's law $v = ki$, in which v is the velocity of seepage and i is the hydraulic gradient. Values of k depend on properties of the fluid, as well as on the porous medium, and reflect any interactions of the fluid with the porous medium such as swelling of soil. The term "permeability" is used in a general sense to indicate the ability of soils to transmit water. In this paper the term "hydraulic conductivity" is used in the specific sense to mean the rate of transmissibility of water through soil in cu in. per sq in. per hr, and usually abbreviated "in. per hr," assuming a hydraulic gradient of unity.

In the past much research work has been done on developing various methods and techniques for determining the permeability of soils. In general, these methods and techniques fall into two types depending on whether the soil was tested "in place" or whether soil samples were transported to the laboratory for testing. Tests made "in place" are generally considered superior for drainage investigations as it is almost impossible to obtain samples and transport them to the laboratory without disturbing them and altering their permeability.

Laboratory tests may be made in either of two ways. An undisturbed core may be obtained with a special core sampler or a disturbed sample may be obtained and taken to the laboratory where it will be dried, reduced to granule size, and then packed into a permeameter tube for testing.

Laboratory-testing methods involve placing soil samples in permeameters where they are subjected to a head of water for a designated period of time. By measuring the amount of water that passes through the sample in a given period of time the hydraulic conductivity is determined.

The method of in-place measurement of hydraulic conductivity most commonly used by the SCS was described by D. Kirkham.[8] This method involves the establishment of test holes usually 4 in. or less in diameter to a depth below the water table. The hole is pumped to a level below the static water level, and the rate at which the water in the hole returns to the original level is measured. The rate of return is a function of and enables computing the hydraulic conductivity.

Two methods of making the tests are used. These are the auger hole method and the piezometer method. Both methods are recommended for use in the SCS.

1. The auger-hole method may utilize holes dug in conjunction with the soil profile investigations. Special holes may be augered for the purpose. The hole is pumped or bailed out several times to flush any puddled-over pores along the wall of the cavity. The water level in the auger hole is allowed to become static following the cleaning process.

To make the test the water level is lowered in the auger hole with a pump or bail bucket. The distance the water level is lowered will depend upon the stability of the soil formation. Where sloughing and caving is a problem a smaller drawdown should be used and a liner of well screen or perforated pipe may be used.

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[6] "Drainage Investigation Methods for Irrigated Areas in Western United States," by W. W. Donnan and G. B. Bradshaw, USDA Tech. Bulletin 1065, 1952.

[7] "Measurement of the Hydraulic Conductivity of Soil in Place," by D. Kirkham, Journal Paper No. J-2505, Iowa Agric. Experiment Sta.

[8] D. Kirkham, Amer. Soc. of Testing Materials, Special Tech. Bulletin 163, 1955.

2. The piezometer method is similar in principle except a watertight pipe is installed with a small cavity below the bottom of the pipe. The hydraulic conductivity is measured from the cavity area.

Typical field charts used for permeability measurements made by the auger hole and piezometer methods are shown in Figs. 3 and 4. The procedures are explained on the charts.

GROUNDWATER INVESTIGATIONS

The purpose of a groundwater investigation is to provide the essential information on the position and fluctuation of the water table at various points in the problem area. This indicates the extent and severity of the drainage problem over the area and provides information for selecting the general type and location of subsurface drains. Information on the position and fluctuation of the water table is obtained by establishing observation wells and piezometers as needed. Artesian pressures and water-flow should be investigated.

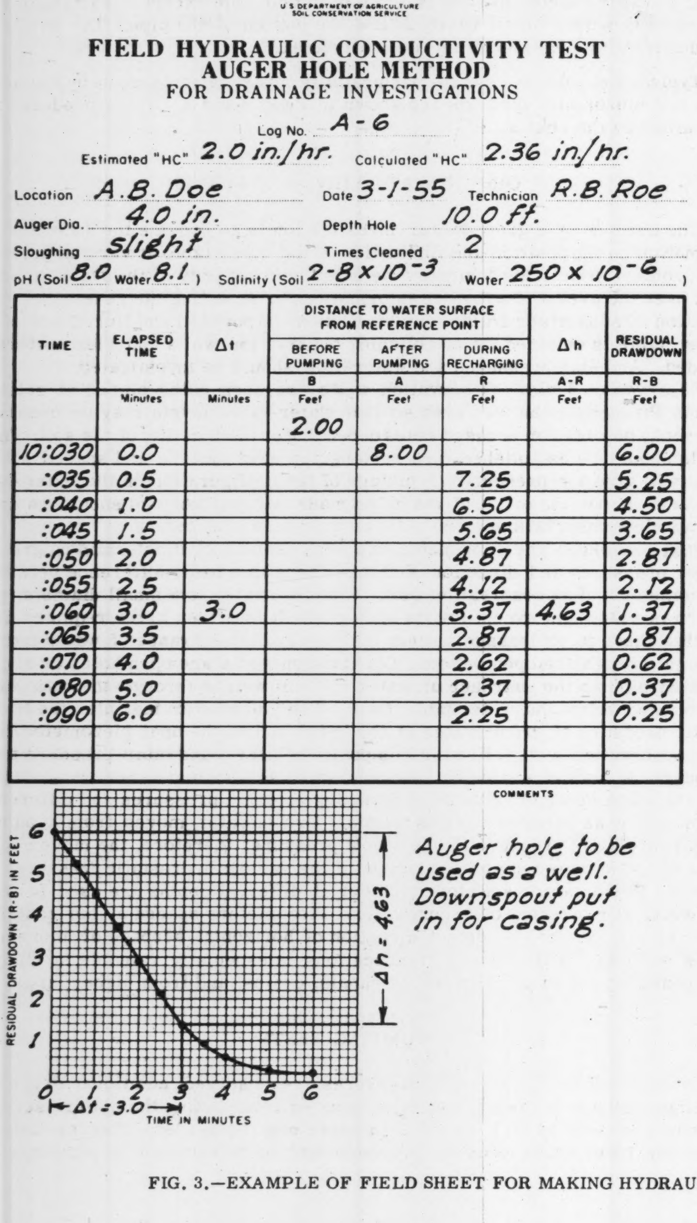
Observation Wells.—Observation wells are open holes placed at selected points throughout the wet area so that water-table levels may be observed. They may be cased or uncased holes depending on the stability of the soil. These wells should be established on a rectangular grid pattern and spaced so that they will give a representative picture of the configuration of the water-table surface. A portion or all of the holes made for soil borings should be cased and used as observation wells.

Piezometers.—The piezometer is a very useful tool in determining groundwater pressures and direction of flow. There is a basic difference between a piezometer and an observation well. The piezometer is a small diameter pipe which is watertight except for its open ends. It is driven to the required depth so that there is no leakage around the pipe and all entrance of water into the pipe is through the open bottom. Observation wells are open holes or of perforated pipe so the entrance of water into the well is through the entire area penetrated below the water table level. The piezometer indicates the hydrostatic pressure of groundwater at the lower end of the open piezometer tube. The observation well reflects a composite of all groundwater pressure to the depth of the well.

The piezometer is valuable in detecting artesian pressures and differences in pressure as between various strata. Groundwater moves from a point of high hydrostatic pressure to one of low pressure; therefore, the movement or flow of groundwater can be determined if the various hydrostatic pressures are known. Piezometers set close together at different depths are used to detect the vertical movement of groundwater. Piezometers spaced at horizontal intervals can be used to detect direction of horizontal seepage or movement. This technique with various ramifications is especially valuable in studying groundwater movement adjacent to canals, drains, and from higher lands.

PUMPING TESTS

Where substantial artesian pressures are observed, additional deeper investigations are needed to locate the aquifer contributing the upward seepage. Borings as deep as 50 ft to 100 ft or more may be needed. The feasibility of pumping from aquifers under pressure may be determined by pumping tests



U S DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

FIELD HYDRAULIC CONDUCTIVITY TEST
AUGER HOLE METHOD
FOR DRAINAGE INVESTIGATIONS

Log No. A - 6

Estimated "HC" 2.0 in./hr. Calculated "HC" 2.36 in./hr.

Location A.B. Doe Date 3-1-55 Technician R.B. Roe

Auger Dia. 4.0 in. Depth Hole 10.0 ft.

Sloughing slight Times Cleaned 2

pH (Soil 8.0 Water 8.1) Salinity (Soil 2-8 x 10^{-3} Water 250 x 10^{-6})

TIME	ELAPSED TIME	Δt	DISTANCE TO WATER SURFACE FROM REFERENCE POINT			Δh	RESIDUAL DRAWDOWN
			BEFORE PUMPING	AFTER PUMPING	DURING RECHARGING		
			B	A	R	A-R	R-B
	Minutes	Minutes	Feet	Feet	Feet	Feet	Feet
			2.00				
10:030	0.0			8.00			6.00
:035	0.5				7.25		5.25
:040	1.0				6.50		4.50
:045	1.5				5.65		3.65
:050	2.0				4.87		2.87
:055	2.5				4.12		2.12
:060	3.0	3.0			3.37	4.63	1.37
:065	3.5				2.87		0.87
:070	4.0				2.62		0.62
:080	5.0				2.37		0.37
:090	6.0				2.25		0.25

COMMENTS

Auger hole to be used as a well. Downspout put in for casing.

FIG. 3.—EXAMPLE OF FIELD SHEET FOR MAKING HYDRAULIC

FIELD HYDRAULIC CONDUCTIVITY TEST
AUGER HOLE METHOD

A knowledge of in place soil permeability is very important in drainage design. Permeability is essential in the design of grids as well as interception drains.

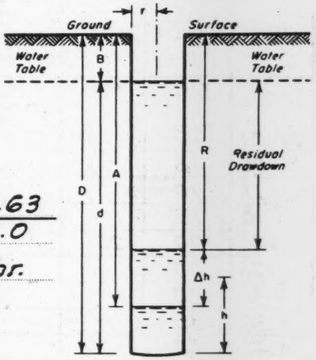
The auger hole method described below is a relatively simple test that can be made with a minimum of equipment and in conjunction with the soil profile investigation. The term hydraulic conductivity is a permeability figure dependent on properties of the groundwater, as well as the soil profile.

The drainage soil profile log holes are used for the permeability tests or a special hole can be augered for the purpose. The hole is pumped or bailed out several times to permit any puddled-over pores along the wall of the cavity to be flushed out by the inseeping groundwater. This flushing process can be accomplished with a pitcher pump or a bail bucket slightly smaller than the auger hole. The water level in the auger hole is allowed to become static following the cleaning process.

TEST: The water level is lowered in the auger hole with the pump or bail bucket. The distance the water level is lowered will be dependent upon the caving and sloughing tendency of the profile. Where sloughing is a problem a smaller drawdown should be used and possibly a liner or screen will be required. The water levels and times of observation are recorded on the form. This time and distance of rise is used in the following Kirkham auger hole formula to calculate the hydraulic conductivity. The depth of water in the auger hole (D-B) should be about 5 to 10 times as deep as the diameter (2r) of the auger hole.

$$HC = 444 \times \frac{r}{Sd} \times \frac{\Delta h}{\Delta t}$$

HC - Hydraulic conductivity in inches per hour
r - Radius of auger hole in feet
S - Function from figure on this page
d - Depth of water in auger hole in feet (D-B)
Δh - Raise of water level in feet in Δt timed interval (A-R)
Δt - Time required to give Δh in minutes
h - Average depth of water in auger hole during test ($D-A+\Delta h/2$)



Auger Hole Profile

r = 0.166

d = 8.0 $HC = 444 \times \frac{0.166}{6.0 \times 8.0} \times \frac{4.63}{3.0}$

h = 4.31

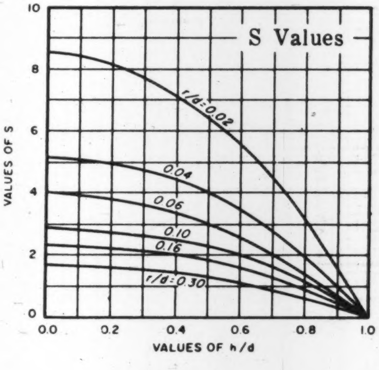
HC = 2.36 in./hr.

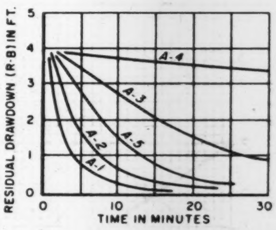
Δh = 4.63

Δt = 3.0

r/d = 0.166/8.0 = 0.02

h/d = 4.31/8.0 = 0.54 S = 6.0





An estimate of the relative hydraulic conductivity between auger holes can be determined by plotting the residual drawdown at various recharge times. The slope of the curve gives an indication of hydraulic conductivity. The steeper the curve the higher the conductivity.

CONDUCTIVITY TEST BY THE AUGER HOLE METHOD

U S DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

FIELD HYDRAULIC CONDUCTIVITY TEST
PIEZOMETER METHOD
FOR DRAINAGE INVESTIGATIONS

Piezometer Number B-7

Estimated "HC" 0.20 in./hr. Calculated "HC" 0.32 in./hr.

Location A. B. Doe Date 6-1-55 Technician R. B. Roe

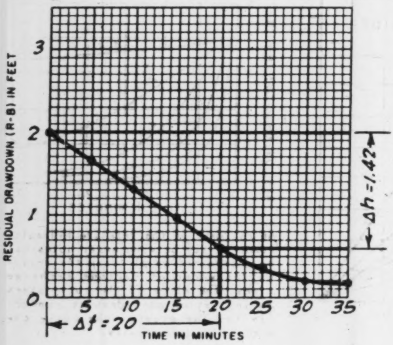
Stratum Thickness 2.0 ft. Texture silt Structure bk.

Depth Piez. 4.2 ft. Auger Dia. 1 3/16 in. Piez. Dia. 1 1/4 in.

Length Cavity 4.0 in. Sloughing None Times Cleaned 3

pH (Soil 8.0 Water 8.1) Salinity (Soil 2×10^{-3} Water 250×10^{-6})

TIME	ELAPSED TIME	Δt	DISTANCE TO WATER SURFACE FROM REFERENCE POINT			Δh	RESIDUAL DRAWDOWN
			BEFORE PUMPING	AFTER PUMPING	DURING RECHARGING		
			B	A	R	A-R	R-B
	Minutes	Minutes	Feet	Feet	Feet	Feet	Feet
			2.00				
11:01	0			4.05			2.05
:06	5				3.70		1.70
:11	10				3.35		1.35
:16	15				3.00		1.00
:21	20	20			2.63	1.42	0.63
:26	25				2.35		0.35
:31	30				2.23		0.23
:36	35				2.15		0.15
:41	40				2.10		0.10



COMMENTS

FIG. 4.—EXAMPLE OF FIELD SHEET FOR MAKING HYDRAULIC

FIELD HYDRAULIC CONDUCTIVITY TEST

PIEZOMETER METHOD

The piezometer method is used to obtain the hydraulic conductivity of a given strata or area in a soil profile. (Hyd. cond. is a permeability figure dependent on properties of the groundwater as well as the soil profile.) This is possible because the hole which is bored into the soil for the conductivity measurement is cased, except for a small cavity at its end. The rate of entry into this cavity is a measure of the hyd. cond. of the soil around the cavity.

EQUIPMENT: A 1½ to 2 inch worm auger with a square bit end is used for the test. Electrical conduit 1¼ to 2 inch inside diameter, sharpened on one end is used as the piez. The auger is ground to about 1/16 inch smaller than the inside dia. of the piez. A driving head on the piez. top prevents damage during driving. An electrical device sounding bell or blow tube can be used to measure the water level. A soil tube jack or piez removal equipment can be used to remove the piez.

METHOD: An auger hole is bored to a depth of 6 inches. The piez. is then driven into the hole about 5 inches with light blows from a maul. The hole is again augered to a depth of 6 inches below the piez. This procedure is continued until the piez. reaches the desired depth. A cavity 4 inches long is carefully augered below the end of the piez. A stop on the auger handle helps make this length precise. The auger should be removed very slowly to prevent sloughing of the cavity wall. A hollow auger or small tube to the auger bit may be required to permit air to break the suction and prevent sloughing of the cavity. The piez. is pumped or bailed out, with a pitcher pump or bail bucket, to permit the pores in the cavity wall to be flushed out. Flushing is repeated until the rate of rise in the piez. is the same as a previous pumping.

TEST: The water level is lowered in the piez. a distance dependent upon the sloughing tendency of the profile. The water levels and times of observations are recorded and used in the following Kirkham Piezometer formula to calculate the hyd. cond.

$$HC = 377 \times \frac{r^2}{c} \times \frac{\Delta h}{\Delta t} \times \frac{1}{A+R-2B}$$

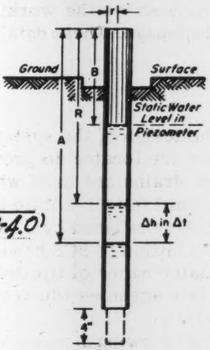
HC - Hydraulic conductivity in inches per hour
r - Inside radius of piezometer in inches
c - Function from figure on this page
Δh - Raise of water level in feet in t timed interval (A-R)
Δt - Time required to give h in minutes
A - Depth to water level at start of test
R - Depth to water level at end of test
B - Depth to static water level in feet

r = 0.63
c = 12.5
Δh = 1.42
Δt = 20
A = 4.05
R = 2.63
B = 2.00

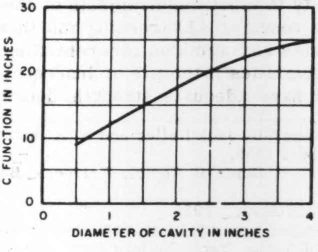
$$HC = 377 \times \frac{0.63^2}{12.5} \times \frac{1.42}{20} \times \frac{1}{(4.05+2.63-4.0)}$$

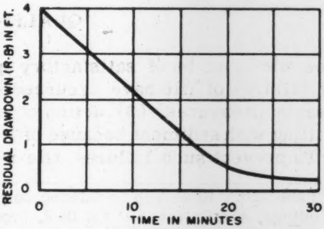
HC = 0.32 in/hr.

Piezometer Profile



C Function for Cavity Four Inches Long





An estimate of hydraulic conductivity can be determined by plotting the residual drawdown at various recharge times. The shape of the curve can also be used in evaluating characteristics of the soil strata.

CONDUCTIVITY TEST BY THE PIEZOMETER METHOD where initial studies indicate that pumping from wells may contribute toward solving the drainage problem.

ANALYSIS OF DATA

After data are collected the analysis includes the following:

1. Water table contour maps. A contour map is used to plot the elevations of the water table at a specific date. The water table contours indicate the direction of flow and head potentials.
2. Depth of water table map. Based on the information which is developed by the water table contour map the depths to water table are shown on another map. This indicates areas with dangerously high water table.
3. Observations well hydrographs. The water table elevations are shown plotted against time. The seasonal fluctuations for critical wells are helpful for determining the source of groundwater.
4. Profile flow patterns for groundwater. These data are plotted on profile paper showing soil layers, groundwater, and hydraulic conductivity. Profile flow patterns are helpful in detecting artesian conditions, seepage from irrigation canals and other sources of underground flow.

Fig. 5 shows the working map completed with water table problems due to canal seepage. These data were obtained by investigations previously described.

DESIGN CRITERIA

The design of the system is established through the investigations described. Drains are located to provide the most effective subsurface drainage. Interceptor drains are used where feasible. Such drains may lower the groundwater from 1/4 to 1/2 mile below the drain. The theory of interceptor drainage has been explained by J. Keller and A. R. Robinson.[9]

In addition to SCS handbooks, previously mentioned, the design, installation, and maintenance of tile drainage systems are covered in publications of federal and state agencies which are available,[10,11,12] and in textbooks published commercially.

The following discussion is limited to some recent developments in the field.

QUALITY OF TILE

The tile must be of satisfactory quality to meet requirements of the site. Many failures of tile have occurred as a result of (1) freezing and thawing; (2) earth pressures; (3) action of sulfates and acids on concrete tiles; and (4) filling with sediment because of irregularities in the tile or inadequate filter. To prevent such failures, tile which have adequate strength, density, or

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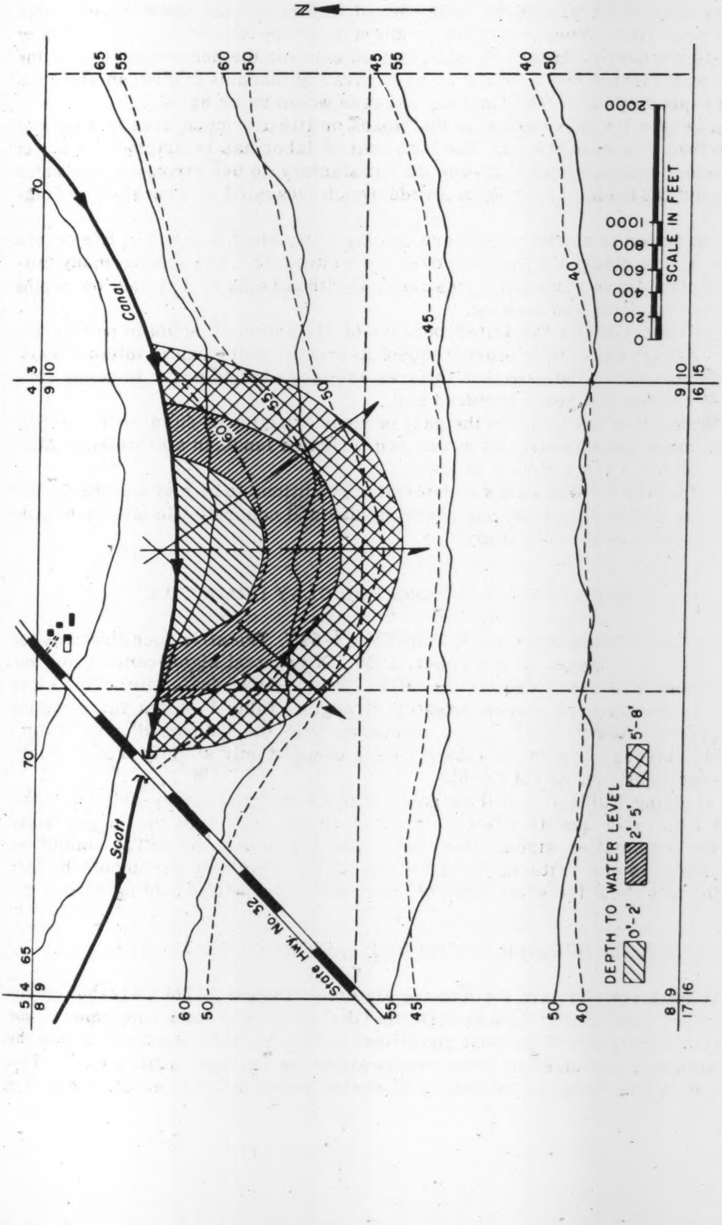
[9] "Laboratory Research on Interceptor Drains," by Jack Keller and A. R. Robinson, Proceedings, ASCE, Vol. 85, No. IR 3, September, 1959.

[10] "Farm Drainage," by Lewis A. Jones, U. S. Dept. of Agric., Farmers Bulletin 2046, October, 1952.

[11] "Water," U. S. Dept. of Agric., Yearbook of Agric., 1955.

[12] "Drainage Investigation Methods for Irrigated Areas in Western United States," by W. W. Donnan and G. B. Bradshaw, U. S. Dept. of Agric., Tech. Bulletin 1065, September, 1952.


N
Canal
Scott
State Hwy. No. 32
DEPTH TO WATER LEVEL
0'-2'
2'-5'
5'-8'
SCALE IN FEET
0
200
400
600
800
1000
2000
65
55
60
50
45
55
40
50
70
70
65
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9
16

FIG. 5.—WORKING DRAWING (CANAL SEEPAGE)

uniformity in size should be used. Soils and soil water should be tested to determine sulfate content or acidity. Concrete tile should be selected based on alkali or acid conditions.

Excessive earth pressures have caused failure of clay and concrete drain tile in deep fills. Formerly, this problem could be solved in most cases by excavating a narrow trench by hand in deep cuts for the deepest portion of the cut. A wider trench would be cut by an excavating machine down to an elevation 2 or 3 ft above grade. The final cut to grade would be by hand.

The reason for this method is that loads on tile are much less in a narrow trench than in a wide trench. The high cost of labor has restricted the use of this method and, generally, it is more satisfactory to use stronger pipe which will resist the loads occurring in a wide trench excavated by a backhoe or dragline.

Frost action is especially severe on clay tile. Much damage to tile occurs by leaving tile stacked in the open over the winter. There have been many failures of clay tile in northern states due to the lines being laid at shallow depths subject to freezing and thawing.

Some concrete tile has failed because of the action of acids in soil or soil water. Failures have been more frequent in organic soils than in mineral soils. Some observations indicate that failures are more apt to occur in sandy soils than in medium and heavy textured soil.

Concrete tile has failed in the past because of action of sulfates on the tile. Such failures occurred chiefly in western irrigated lands, in southwestern Minnesota, and in a few counties in Iowa.

Satisfactory investigations to determine conditions to be met and the proper use of the ASTM specifications covering clay and concrete tile should help eliminate these causes of failure.

ASTM SPECIFICATIONS FOR CLAY DRAIN TILE

Important changes were made in ASTM C4-59T, Tentative Specifications for Clay Drain Tile, issued in November, 1959. Of greatest significance in the establishment of a new class of tile called "heavy-duty" drain tile. This class of tile is designed to provide greater strength. Table 1 shows the crushing strengths of "heavy-duty" tile as compared with the "standard" and "extra-quality" classes. The latter classes were changed only slightly from the former specifications, ASTM C4-55:

In studying Table 1 it will be noted that strengths of heavy-duty tile compared with extra quality tile have been increased more for the larger sizes than for the smaller sizes. The reason for this is that the ASTM committee established as one criteria that all sizes of the heavy duty tile should be able to withstand about the same depth of cover under projection loading conditions.

ASTM SPECIFICATIONS FOR CONCRETE DRAIN TILE

The American Society for Testing Materials issued ASTM C412-58T, Tentative Specification for Concrete Drain Tile, which has come into general use within the last year. The most significant change in this specification was the inclusion of a new class of concrete tile known as "special-quality tile." This class of tile is designed especially to resist action of acid and alkalies. The object is to obtain a dense tile made from well graded aggregate which minimizes the leakage of water through the walls of the tile. These specifications provide for a hydrostatic test as an alternate test to a minute absorption test for special quality tile to insure a minimum of leakage through walls.

GUIDE FOR USE OF CONCRETE TILE IN ACID AND SULFATE SOILS

The Soil Conservation Service has prepared a guide for use when working with the three classes of drain tile shown in Table 2. These limits have been worked out by drainage engineers of the SCS in cooperation with P. W. Manson of the Agricultural Engineering Department, University of Minnesota. The careful application of limits shown in Table 2 will greatly improve the durability of concrete pipe installed in acid and sulfate soils.

TABLE 1.—CRUSHING STRENGTHS OF CLAY DRAIN TILE AS GIVEN IN ASTM C4-59T

Tile Size	Minimum Crushing Strength (average of five tile)		
Diameter, in in. (1)	Standard, in lb per ft (2)	Extra quality, in lb per ft (3)	Heavy duty (new class), in lb per ft (4)
4 to 6	800	1,100	1,400
8	800	1,100	1,500
10	800	1,100	1,550
12	800	1,100	1,700
15	870	1,150	1,980
18	...	1,300	2,340
21	...	1,450	2,680
24	...	1,600	3,000
30	...	2,000	3,590

BLINDING TILE AND FILTER REQUIREMENTS

Most of the tile drains in the Cornbelt States have adequate protection against rapid silting by proper blinding of drain tile with topsoil. Sod and topsoil having a high organic content make excellent blinding material to filter water which enters the tile. Contractors have generally perfected the art of blinding so that little trouble occurs and tile may be expected to have an effective life of 20 to 30 yr or more.

However, there are some areas where silting of tile lines is a serious problem. Generally, this occurs where fine sand and coarse silt particles having a low cohesion flow into the tile. Where tile is laid on a grade to produce a design velocity of about 1.5 fps or more, the velocity is high enough to wash out much of the sediment. Laying tile on grades producing such velocities is a practical solution in some locations.

Progress has been made in solving the sediment problem by design of adequate gravel or sand filters, which are widely used. If the filter material is too coarse the base material flows through the filter. If the filter material is too fine the filter impedes the movement of water into the tile. Research by several agencies has established the desirable particle size limits for gravel filters. Frequently, pit run gravels are well graded and make an excellent filter material. Often it is desirable to mix sand or gravel from two sources to obtain a filter material having satisfactory size distribution.

Mechanical analyses of base filter material are made and gradation curves are plotted. Recommended standards adopted by the Soil Conservation Service are based on research of the Corps of Engineers, Bureau of Reclamation, and others. They are as follows:

TABLE 2.—GUIDE FOR USE OF CONCRETE TILE IN ACID AND SULFATE SOILS

(a) Acid Soils

Class of tile ASTM C412-58T[a]	Lower Permissible limits of pH values[b]	
	Organic and sandy soils	Medium and heavy-textured soils
Standard quality	6.5	6.0
Extra quality	6.0	5.5
Special quality[c]	5.5	5.0

(b) Sulfate Soils

Permissible maximum limit of sulfates (Na or Mg) singly or in combination[d]	ASTM class of tile and cement[e]
(Parts per million)	
7,000	Special quality, type V or type II cement[f]
3,000	Extra quality, type V or type II cement[f]
1,000	Standard quality, any type cement

[a] Where manufacturer or seller desires to furnish tile of a class which fails to qualify under limits shown in this table, he will be required to furnish proof that tile recommended will give satisfactory service life.

[b] Figures given represent lowest readings of pH values for soil water or soil at tile depth. Values to be confirmed by additional tests where desirable.

[c] Obtain high strength, low absorption tile where available. See Minn. Agric. Experment Sta., Bulletin 426, "Making Durable Concrete Draintile," by P. W. Manson and D. G. Miller.

[d] Highest reading of sulfates for soil or soil water at tile depth. Values to be confirmed by additional tests where desirable.

[e] ASTM specifications C412-58T and C150-55. Where manufacturer or seller desires to furnish tile or cement of a class which fails to qualify under limits shown in this table he will be required to furnish proof that tile recommended will give satisfactory service life.

[f] Type II cement to be of high sulfate resistance. The calculated tricalcium aluminate (C_3A) compound of the cement should be less than 5.5%.

Sand or sand-gravel filters for drain conduits should be constructed of hard, durable sand or sand-gravel mixtures.

Sand or sand-gravel filter material should be selected or mixed in accordance with the gradation of the soil mass in and beneath which the conduit is to be placed. The following criteria are recommended in designing the filter:

(1) Maximum size of Material = 2 in.

(2) $\frac{D_{50} \text{ size of filter}}{D_{50} \text{ size of soil}}$ = 12 to 58 (D_{50} size is that size for which 50% of the

material by weight is smaller; D_{15} is that size for which 15% by weight is smaller; etc.)

(3) $\dfrac{D_{15}\text{ size of filter}}{D_{15}\text{ size of soil}} = 12 \text{ to } 40$

(4) Not more than 5% of filter material should pass the No. 200 sieve.

(5) $\dfrac{D_{15}\text{ size of filter}}{D_{85}\text{ size of soil}} = \text{less than } 5$

(6) The gradation curve of the filter material should be approximately parallel to that of the soil.

(7) The D_{85} size of filter should be no less than 1.2 times the maximum joint spacing between drain tile or other pipe sections.

(8) Minimum thickness of filter around drain conduit should be 3 in.

In a recent publication G. Kruse[13] reports on experiments simulating radial flow into an irrigation well with the flow reversed at intervals to determine criteria for stability of uniform and non-uniform aquifers and gravel packs. The uniformity coefficient (C_u) was defined as the ratio of the D_{60} size to the D_{10} size of granular materials. The pack-aquifer (P-A) ratio was defined as the D_{50} size of the gravel pack to the D_{50} size of the aquifer. Test results indicated the following values as upper limits of the pack-aqufier ratios, if a stable filtering action was to be maintained.

Aquifer	Gravel Pack	Limiting P-A ratio
Uniform	Uniform	9.5
Non-Uniform	Uniform	13.5
Uniform	Non-Uniform	13.5
Non-Uniform	Non-Uniform	17.5

These experiments do not simulate conditions found in draintile but indicate advantages of non-uniform gradation of materials and of a low P-A ratio. They indicate the desirability of further research to determine criteria for stable filter conditions for drain tile.

Glass fiber material has found considerable use as a tile filter in recent years. This material was tested by the Agricultural Research Service at Ft. Collins, Colo. The preliminary results showed that the material will restrain particles in the coarse silt, fine sand, and larger sizes from entering the tile. The strength of the material having cross-fiber bracing is more satisfactory than that reinforced by longitudinal fibers. This material is finding increasing use.

BITUMINIZED FIBER PIPE

Also of interest is the use of other materials for drain tile. Bituminized fiber drain pipe has been used in several states for drain tile. Most installations have been of perforated pipe, 4-in. size, and lengths of 8 ft. The pipe is light and is on a competitive basis with other pipe in many areas in the Northeast, especially where the freight is an important cost factor.

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[13] "Selection of Gravel Packs for Wells in Unconsolidated Aquifers," by Gordon Kruse, Colorado State Univ. Experiment Sta., Tech. Bulletin No. 66, March, 1960.

The specifications for bituminous fiber drain pipe are the current U. S. Department of Commerce Commercial Standards (CS116 - 54) modified as follows:

1.	Pipe size (minimum inside diameter, in in.)	4	5	6
2.	Minimum pipe wall thickness, in in.)	0.25	0.32	0.34
3.	Ultimate crushing strength, in lb per ft of pipe by 3-edged bearing test	800	800	800
4.	Rows of perforations (Perforations in row on 3 in. centers. Perforations shall be omitted when specified for use as conduit on steep slope.)	2	4	4
5.	Diameter of perforations (+ or - 1/16 in. Standard 5/8 in. perforations may be specified for some organic soils when recommended in local drainage guides.)	5/16	5/16	5/16
6.	Radial spacing of holes for two-row perforations shall be 105°. Radial spacing of holes for 4-row perforations shall be 35°, 120° and 155°, successively, from the first row.			

PLASTIC PIPE

There has been some installations of plastic drain pipe and considerable interest in the use of this material. So far installations have been few in number. The cost and service life of this material will be determined by experience and will influence the extent of its use.

PORTABLE TILE TESTING MACHINE

SCS technicians have recently used a portable testing machine to enable field tests of tile to insure that they meet specifications. A machine developed by SCS technicians weighs less than 100 lb and consists of a two-ton hydraulic jack, a ram, gage, and frame of steel beams. Light weight portable testing machines are also available commercially.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3141

COMPUTER APPLICATIONS IN GROUNDWATER HYDROLOGY

By Joseph Foley,[1] A. M. ASCE

SYNOPSIS

The Theis equation is adapted for computer solution of four types of problems inherent in many industrial well-water developments. These types are (1) drawdown computation, (2) aquifer constants "T" and "S," (3) well spacing, and (4) capacity of a well system. Objectives, equations, computation sequence, print-out of results and computer time are outlined for each problem.

INTRODUCTION

Computer programs have been developed for the solution of four general types of hydrological problems. The objective in preparing these programs is to give the groundwater hydrologist a rapid and accurate means of solving these types of problems for any conditions specified or for any alternate value of a variable or variables. An alternate objective is to provide prepared mathematical solutions which engineers, who may have only a limited knowledge of the details of solving these types of problems, may use with confidence.

Whereas all of the variables involved in ground water hydrology may finally enter into the overall problem, those of main interest to the groundwater hydrologist are the two that define the characteristics of the aquifer; namely, the coefficient of storage, S and coefficient of transmissibility, T. In determining these, the usual procedure is to conduct an aquifer test, making observations of all the measurable variables. These include pumping rate, Q in gallons per minute, total drawdown, s, time, t, and the distance r to each observation well. Once this has been done, the aquifer characteristics can be

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Note.—Published essentially as printed here, in September, 1960, in the Journal of the Irrigation and Drainage Division as Proceedings Paper 2598. Positions and titles given are those in effect when the paper or discussion was approved for publication in "Transactions.

[1] Cons., Engrg. Service Div., E. I. du Pont de Nemours and Co., Wilmington 98, Del.

determined by either (1) graphical solution using the "Type Curve" devised by Theis,[2] or (2) graphical solution devised by C. E. Jacob[3] on the basis of a modification of the non-equilibrium equation originated by Theis.

Generally, these two methods have been the most widely used in groundwater hydrology. Each involves the collection of numerous observations at frequent and brief intervals of time during a pumping test at essentially constant rate. In either case, it is necessary to pump a considerable length of time to get a large number of observations for plotting on appropriate graph paper. Data obtained from these plottings are then used in modified mathematical equations to compute the necessary aquifer constants. Once these have been determined, additional computation can be made to determine other items of importance with respect to the groundwater hydrology and the well water development. Supplementary plottings and computation are usually made for the study of such variables as long-term drawdown, increased pumping rates, variation of distances between permanent wells, the effect on well yield of interferences between wells, and the total number of wells required in the final installation to develop the full amount of well water desired. This work requires much valuable time which can be significantly reduced by utilization of a computer.

DRAWDOWN COMPUTATIONS

Program.—Drawdown computations involves drawdown in an observation well, at any time, by the non-equilibrium formula with known or assumed values of aquifer constants and pumping rate. This computer program solves for the drawdown "s" in an observation well by the Theis non-equilibrium equation.

$$s = \frac{114.6\ Q}{T}\left[-0.577216 - \ln u + \left(u - \frac{u^2}{2\cdot 2!}\ \frac{u^3}{3\cdot 3!}\cdots\right)\right] \ldots (1)$$

in which $u = \frac{1.87\ r^2\ S}{T\ t}$ and $\ln = \log_e$

The computation is based on the fact that all factors in the equation have assigned constant values except s and t. In the solution of the problem, values of t are selected for which values of the drawdown s are to be computed. The objective of preparing a program for this type of solution is to provide a rapid and accurate method for computing drawdowns for a variety of conditions. It eliminates the necessity of resorting to graphs, tables or formulas.

The practical uses for this program might include the following:

1. Forecasting performance of a number of observation wells during an intended pumping test based upon assumed values of T and S.
2. Readjusting the spacing of observation wells to insure significant drawdown observations within the scope of an intended pumping test.

---

[2] "The Relation Between the Lowering of the Piezometric Surface and the Rate and Duration of Discharge of a Well Using Ground Water Storage," by C. V. Theis, Transactions, Amer. Geophysical Union, pt. 2, 1935, p. 519.

[3] "Drawdown Test to Determine Effective Radius of Artesian Well," by C. E. Jacob, Transactions, Vol. 112, 1947.

3. Determining the limit of time for test pumping, beyond which the rate of drawdown per day in any observation well may be insignificant with respect to other factors affecting water level.
4. Conducting a rapid check on the accuracy of computed values of T and S by computing drawdowns for comparison with test observations.

In the solution, values are assigned to S and T. Actual values for the type of formation may be available from other tests. The values may also be estimated based upon known geology, possibly from geological test holes drilled at the site to determine the depth, thickness and extent of water-bearing formations. Normally, the value of r is known for each observation well as is Q, the intended pumping rate. The only unknowns than are s and t. Therefore, s may be solved for any specified value or series of values of t.

Computation Sequence.—Generally, the computer procedure involves the operations of computation, control tests on magnitude of values, placing values in working storage, print-out of desired values and recomputation for successive values of t. The sequence that follows illustrates the computer procedure devised by J. L. Hertig.

Compute "u" - Test
Compute Series Term - Test
Add Series Terms
Subtr Ln "u"
Subtr 0.577216
Compute "s"
Compute Δs
Print Out
Test "t" - Test Δs - Test Counter - Stop Comp.
Recompute for next
value of "t"

Control Tests.—The program contains the following control tests of particular significance with relation to limiting the computations to the proper hydrologic scope.

For series term error.—As long as the value of u exceeds the factorial number in the denominator, the value of the series term is increasing. After this the value decreases and reaches a value beyond which it has no significant effect upon the computations. An error factor has been specified such that the computer will stop computing series terms when the value of a series term becomes less than the error factor. This factor is an item of input data and can be changed if desired.

For rate of drawdown.—The main portion of computation in the computer program involves computing the drawdown for each successive day of pumping. A control figure has been included as a part of input data which will stop this portion of the computations when the increment of drawdown from one day to the next is less than the specified control figure. This control figure may be assigned any value desired.

Data Print-Out.—Fig. 1 is a sample of the print-out obtained as computations are completed. Note that it first prints the assigned input data values of T, S, r and Q. This is then followed by a tabulation listing first the values of t as specified, then the corresponding computed values of the drawdown s. This is the actual print-out as made by the computer. It is referred to as floating decimal form. In the floating decimal form, the decimal is always printed in the same place as shown. The two digit number immediately to the right of the seven digit number indicates correct location for the decimal point with reference to the printed location. The two digit number specifies the number of places which the decimal is to be moved. This movement is to the left if followed by a minus sign, otherwise to the right. Fig. 2 is the same data edited in fixed decimal form and is shown as a matter of information.

Note that the computer always prints out a seven-digit number as in Fig. 1 irrespective of the number of significant figures usable as shown in Fig. 2. Also, computers such as the LGP-30 (that used by the author), which must convert a decimal number to a binary floating point for internal processing, are subject to a tiny error in conversion called truncation error. For example, the first value of t = 0.354 is printed by the computer in Fig. 1 as 0.3539999.

During the process of computation, numerous values computed are placed in working storage in the computer. For any given value of "t," the values placed in working storage may be retrieved and printed if desired. For example, Fig. 3 is an alternate print-out of data from the same problem as Fig. 1. The headings of the various columns will be recognized as factors which are of specific interest in most groundwater computations. The computation and print-out of each line of data shown in these figures requires only about 30 sec.

```
    GROUND WATER CALCS.

T                .1890000   05
S                .3849999   03-
r                .2030000   04
Q                .8250000   03

t                s
.3539999   00    .3177059   01
.5000000   00    .4363727   01
.7079999   00    .5695532   01
.1000000   01    .7129710   01
.2000000   01    .1022758   02
.3000000   01    .1213270   02
.4000000   01    .1350636   02
.1000000   02    .1797221   02
.1000000   03    .2941994   02
.1000000   04    .4093049   02
```

FIG. 1.—COMPUTER PRINT-OUT (FLOATING DECIMAL FORM)

```
    GROUND WATER CALCS.

T        =       18,900
S        =      .000385
r        =        2030
Q        =         825

t                  s
      0.354      3.18
      0.500      4.36
      0.708      5.70
      1.00       7.13
      2.0       10.23
      3.0       12.13
      4.0       13.51
     10         17.97
    100         29.42
   1000         40.93
```

FIG. 2.—EDITED PRINT-OUT (FIXED DECIMAL FORM)

Scope of Problem.—In this program, the scope of the computations is limited only by the number of values of t for which the hydrologist desires values of drawdown. For purposes of computation, these values of t are divided into three classifications.

1. Values of t less than one day: Any number of values of t up to eighteen can be specified. These individual specific values of t are entered as input data in order of increasing magnitude. The computer will take them in this order and compute the value of s corresponding to each.
2. Values of t for whole days: After completing the computations for all values of t less than one, the computer will begin calculating drawdowns for

successive days until the rate of drawdown from one day to the next becomes less than the control figure as explained previously. No input data are required for this step in the program.

3. Long term values of t: When the computer has completed the computations in step 2, it then goes to the computation of drawdowns for specific long-term values of t. Provision has been made in the program to include as many as nine specific long-term values for t. When the computer has completed computations of the specified long-term values of t, it will automatically stop and reset itself to receive alterations to the input data.

Time.—An estimate of the total amount of time involved in completely handling a problem containing ten values of t would show the preparation for the initial run to involve 15 min for input data sheets, 5 min for checking computer sub-routines, 1 min for loading program tape, 3 min for loading input data, and 4 min for the computer accuracy check. The operation (involving computation and print-out) takes 5 min making a total time of initial run of 33 min. The supplementary runs involve 2 min to prepare and alter data, and 5 min for operation - computation and print-out producing a total time of 7 min.

```
GROUND WATER CALCS.
T                .1890000   05
S                .3849999   03-
r                .2030000   04
Q                .8250000   03

t                u                W(u)             s                Δs               R
.3539999   00    .4434343   00    .6351095   00    .3177059   01    .3177059   01    .2283298   04
.5000000   00    .3139514   00    .8723302   00    .4363727   01    .1186668   01    .2713602   04
.7079999   00    .2217171   00    .1138564   01    .5695532   01    .1331804   01    .3229072   04
.1000000   01    .1569757   00    .1425263   01    .7129710   01    .1434179   01    .3837612   04
.2000000   01    .7848786   01-   .2044543   01    .1022758   02    .3097873   01    .5427203   04
.3000000   01    .5232524   01-   .2425385   01    .1213270   02    .1905119   01    .6646939   04
.4000000   01    .3924393   01-   .2699986   01    .1350636   02    .1373657   01    .7675225   04
.1000000   02    .1569757   01-   .3592731   01    .1797221   02    .4465850   01    .1213559   05
.1000000   03    .1569757   02-   .5881188   01    .2941994   02    .1144773   02    .3837612   05
.1000000   04    .1569757   03-   .8182203   01    .4093049   02    .1151056   02    .1213560   06
```

FIG. 3.—DRAWDOWN COMPUTATIONS ALTERNATE PRINT-OUT

AQUIFER CONSTANTS FROM TEST DATA

Program.—Aquifer constants from test data involved solution for coefficients of transmissibility and storage using data obtained during an aquifer pumping test. This computer program has been designed to take successive observations of drawdown and time and compute values of S and T that fit the Theis equation.

With an electronic computer a mathematical solution of the Theis equation can be obtained in a relatively short time with a minimum of two observations of drawdown and time. A series of values of S and T can also be obtained, each related to specific observations.

Unlike the graphical methods, numerous observations immediately after pumping starts are not necessary. Relatively few observations, several hours after pumping starts probably will constitute the most useful computer data, in the majority of cases. If a pumping test runs longer than one day, a few observations each day may be adequate for computer purposes.

Major advantages of the computer solution are as follows:

1. Elimination of the hectic pace at the beginning of a pumping test to obtain numerous drawdown and time observations.
2. Reduction in the quantity of data, the number of recorders and the personnel required to obtain observations.
3. Elimination of the need for plotting, together with the need for estimating match points and/or slopes.
4. Replacement of the single average S and T obtained graphically with a series of specific S and T values.
5. Attainment of valid values of S and T regardless of the rates of change of drawdown with time.
6. Reduction of time required to process data from numerous observation wells.
7. Utilization of fragmentary data from interrupted well tests to obtain usable results.

This particular program has been prepared to be run in two steps. Step 1 is identified as the short computation. It is a direct rapid solution that will give approximate values of T and S for use as trial values in the next step. Step 2 is identified as the long computation. It is a trial and error solution for T and S in the Theis equation.

In Step 1 a modified equation (normally used for large values of t only) is used. Values of drawdown and time in days after start of pumping are entered in sequence as data. The computer uses the values of s and t in sequence and in a pre-selected system of "pairs." It computes values of "T" from

$$s_2 - s_1 = \frac{114.6\ Q}{T} \ln \frac{t_2}{t_1} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

It also computes "t_o" and then computes values of S from

$$S = \frac{0.3\ T\ t_o}{r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

Since these are straightforward computations, they can be done very fast. From the results obtained, selected answers are used as initial trial values in the next step.

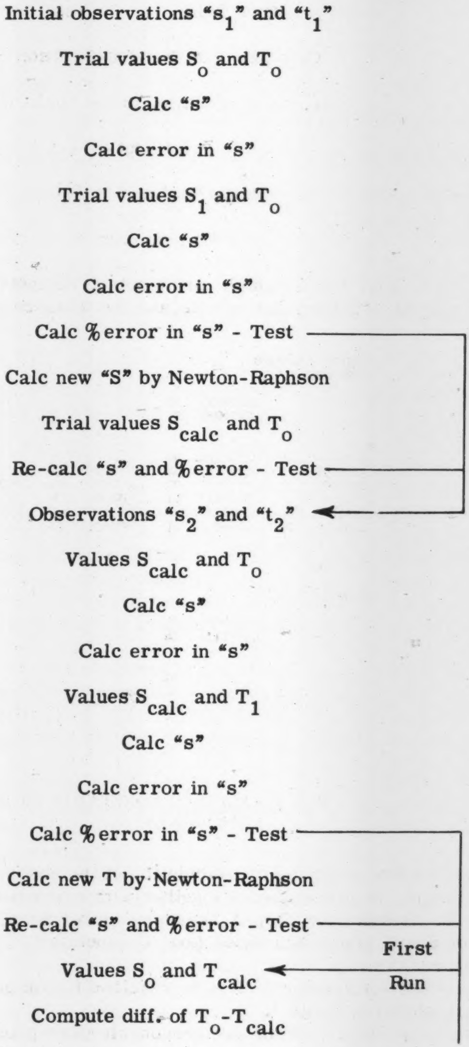
In Step 2, the procedure used for computer solution was devised by J. W. Blakemore and R. C. Haines. It was applied to the general purpose computer by the writer. These procedures involve initial substitution of approximate values for S and T, computation of the resulting error in the computer drawdown s, and finally convergence to the actual "S" and "T" by a modified Newton-Raphson technique.[4]

Computation Sequence.—In the solution, the computation of s is by the method previously described. Generally, the same limitations and controls apply. As previously explained, the solution is fundamentally a trial and error method.

---

[4] "Introduction to Numerical Analysis," by F. B. Hildebrand, McGraw-Hill Book Co., Inc., New York, 1956, p. 447.

A direct mathematical solution is impossible. A trial and error solution of this magnitude is practical only because of the speed with which a computer can operate. An outline of the computation sequence in Step 2 is as follows:



Initial observations "s_1" and "t_1"

Trial values S_o and T_o

Calc "s"

Calc error in "s"

Trial values S_1 and T_o

Calc "s"

Calc error in "s"

Calc % error in "s" - Test

Calc new "S" by Newton-Raphson

Trial values S_{calc} and T_o

Re-calc "s" and % error - Test

Observations "s_2" and "t_2"

Values S_{calc} and T_o

Calc "s"

Calc error in "s"

Values S_{calc} and T_1

Calc "s"

Calc error in "s"

Calc % error in "s" - Test

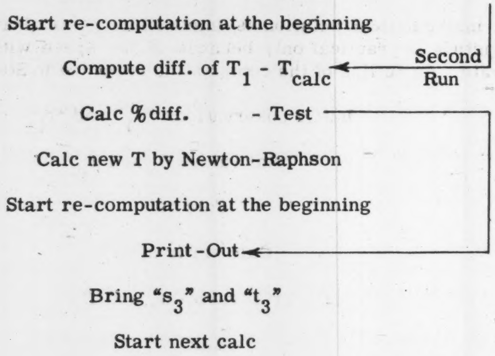
Calc new T by Newton-Raphson

Re-calc "s" and % error - Test

First Run

Values S_o and T_{calc}

Compute diff. of T_o-T_{calc}



Start re-computation at the beginning

Compute diff. of $T_1 - T_{calc}$

Second Run

Calc % diff. - Test

Calc new T by Newton-Raphson

Start re-computation at the beginning

Print-Out

Bring "s_3" and "t_3"

Start next calc

Print-Out.—Fig. 4 is a sample print-out of computer results. The actual test flow rate Q in gallons per minute, and the distance to the observation well r are printed first followed by a tabulation of the observed values of drawdown, s, and time, t, in proper order together with computed corresponding-values of S and T. Results for Step 1 are shown under Short Computation in Fig. 4. Those for Step 2 are shown under Long Computation of Fig. 4 and are the final computed results.

```
AQUIFER CONSTANTS
SHORT COMP

Q                .8250000   03
r                .2030000   04

s                t                   S                   T
.3060000   01    .3559999   00
.9189999   01    .1750000   01    .2874686   03-   .2456073   05
.1023000   02    .2000000   01    .3072679   03-   .2141621   05
.1213000   02    .3000000   01    .3352084   03-   .2001697   05
.1261000   02    .3540000   01    .3065296   03-   .2180964   05
.1351000   02    .4000000   01    .3971055   03-   .1809220   05

LONG COMP

AQUIFER CONSTANTS

Q                .8250000   03
r                .2030000   04

s                t                   S                   T
.3060000   01    .3559999   00
.9189999   01    .1750000   01    .4020069   03-   .1979900   05
.1023000   02    .2000000   01    .4162171   03-   .1763371   05
.1213000   02    .3000000   01    .4272539   03-   .1764134   05
.1261000   02    .3540000   01    .3815286   03-   .1973939   05
.1351000   02    .4000000   01    .4313536   03-   .1769618   05
```

FIG. 4.—AQUIFER CONSTANTS FROM TEST DATA

Time.—The total amount of time required to compute values of S and T in Fig. 4 are shown in Table 1.

These time requirements are reasonable when processing the data from a single observation well, or when using Step 1 only for a number of observation wells for which provision has been made in the program. On the other hand,

TABLE 1

Initial Run:	Time, in minutes	
	Step 1 Only	Steps 1 & 2 Combined
Preparation;		
Input data sheets and tapes	15	15
Check computer sub routines	5	5
Load program tape	3	3
Load input data	2	2
Computer accuracy check	1	17
Operation - Computation and print	5	30
Total time - Initial run	31	72
Supplementary Runs:		
Preparation; Data Sheets & Tapes	5	5
Load Data	2	2
Operation - Computation and Print	5	30
Total time	12	37

when data from numerous observation wells are to be run by the long computation, too much time is involved on the computer used. Other computers, that are faster at this operation shall be used in the future.

WELL SPACING

Program.—Well spacing involves the most economical spacing for a series of pumping wells based on consideration of the factors effecting both hydrology and cost. Recognizing that spacing a series of wells is a compromise between hydrology and total annual cost, this computer program was prepared for the combined solution of the nonequilibrium formula and the cost of a well system. The cost includes the complete cost of well, pump, motor, starter, pipeline between wells, power line and power. The hydrologic factors required are Q, specific capacity, S, T, maximum allowable drawdown s, and a value of t beyond which change in drawdown is relatively insignificant.

This program uses the Theis nonequilibrium equation to compute interferences and then develops the most economical number, pumping rate, and uniform spacing for a line of wells. The program as now prepared will handle up to nine wells of equal capacity.

The program starts by computing the minimum number of wells required to pump the specified total quantity of water. It computes spacing for these wells to provide minimum interference and to allow maximum utilization of the total allowable drawdown for pumping based upon specific capacity. It also computes the size of pipeline required for total output of well field. It then computes the total annual cost for the well system. Next the number of wells is increased by one and computations are repeated. The computer will continue to compute and print-out successive combinations of wells, pumping rates, interferences, spacings and costs in order of increasing number of wells. As the number of wells is increased, the spacing is decreased significantly, resulting in a lower investment and total annual cost. This will continue until the cost of adding wells exceeds the saving of reduced spacing. At this point the decreasing cost is reversed and an increase in total annual cost is obtained. The computer then stops.

The total drawdown was defined as

$$s = \frac{Q}{\text{Sp. Cap.}} + \Sigma s_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which s_1 = the drawdown due to interference from any single well, is defined by

$$s_1 = \frac{114.6\ Q}{T} [\ 0.577216 - \ln u] \quad \ldots\ldots\ldots\ldots (5a)$$

This applies if short term values of t are not used. In this problem we are only concerned with relatively long term values of t. This equation can be rewritten as

$$s_1 = \frac{114.6\ Q}{T} \left[\ln \frac{T\,t}{1.87\ S} - 0.577216 - \ln r^2 \right] \quad \ldots\ldots (5b)$$

Letting $A = \frac{114.6\ Q}{T}$, and $B = \ln \frac{Tt}{1.87\ S} - 0.577216$ then

$$s_1 = A \left[B - \ln r^2 \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (5c)$$

For a series of wells in a line, the distance from the central well to wells on each side is r, 2r, 3r, 4r, and so on, where r is the uniform spacing between all the wells. Let N equal the number of wells and Σ series the sum of ln of the multipliers for r, then the final equation to be solved is

$$\ln r = \frac{B}{2} - \frac{\Sigma s_1 + A\ [\ \Sigma \text{ series}]}{2\ (N-1)\ A} \quad \ldots\ldots\ldots\ldots (6)$$

In developing this program, consideration was given to the fact that nearly all industrial well-water systems are expanded by installation of additional wells in the future. Allowance is made for both present and future conditions in these computations by using the central well or wells as the "control" in computing the total drawdown due to interference. This figure is applied equally to all wells, even though it is recognized that, initially, the end wells will have significantly less interference. In this way, a uniform spacing for all present and future equal-capacity wells is obtained.

A maximum of nine wells in line was selected because this was adequate for interference computations on the central well of the systems considered in the initial problems. The computer has adequate capacity to handle much larger systems if necessary. It is a relatively simple job to rewrite the program when a system must consist of more than nine equal-capacity wells. An alternate procedure would be to simply prorate that portion of the total quantity of water than can be produced from a system of nine wells or less and then compute the most economical individual well capacity and spacing. This same capacity and spacing may be sufficiently accurate to apply to all wells in the system.

Computation Sequence.—As explained previously, the first step in developing this program was to rearrange the Theis equation for solution of r for any known t and known values of Q, S, T, specific capacity and number of wells. Using the computed values of r, the investment and annual cost related to distance are computed and totaled. The outline of a sequence used in the computer solution of this problem is as follows.

Calc Min. No. Wells

Calc DD due to Sp. Cap.

Calc allowable Σs_1

Calc B/2

Calc A

Calc Σ Series

Calc ln r and r

Calc Cost of Power

Calc Well Invest.

Calc Power Line Invest.

Calc Pipe Diam. & Invest.

Calc Annual Fixed Chgs.

Calc Power for Pumping

Calc Total Annual Cost

Print-Out

Test Annual Cost

Increase N by 1

Re-compute

Fig. 5 is a print-out of results of a sample problem. The first tabulation is a listing of pertinent input data required, including both the hydrologic data and the unit cost data. The next tabulation gives the computed results starting with the computed minimum number of wells and adding one well each time until the variable investment (TOT INV) and annual cost (C) start to increase. At this point computation automatically stops and the computer resets itself to receive new data.

Fig. 6 is a plotting of these results, showing that the minimum portion of the cost curve is quite flat. The results must, therefore, be examined to select

the most favorable spacing that truly represents a significant economic saving in relation to the next larger spacing. One refinement in this program would be to stop the computation when a significant reduction in spacing results in only a minor reduction in cost.

```
WELL SPACING

S              .3849999   03-
T              .1890000   05
t              .3650000   03
s              .1000000   03
Sp Cap         .7399999   01
GPM            .2000000   04
Well $ ea      .2000000   05
Pipe $/"/ft    .1000000   01
Cable $/f      .4000000   01
$/KWH          .5999999   02-

N                 Q                 d                 Σsi               Tot Inv           r                 C
.3000000   01    .6666666   03    .1400000   02    .9909911   01    .1490590   07    .3973861   05    .1520110   06
.4000000   01    .5000000   03    .1400000   02    .3243243   02    .6085870   06    .9788649   04    .6380983   05
.5000000   01    .4000000   03    .1400000   02    .4594594   02    .4497922   06    .4858226   04    .4793058   05
.6000000   01    .3333333   03    .1400000   02    .5495495   02    .3848225   06    .2942472   04    .4143369   05
.7000000   01    .2857142   03    .1400000   02    .6138948   02    .3674939   06    .2106428   04    .3970139   05
.7999999   01    .2500000   03    .1400000   02    .6621622   02    .3606139   06    .1592174   04    .3901338   05
.8999999   01    .2222222   03    .1400000   02    .6996996   02    .3658629   06    .1290715   04    .3953829   05
```

FIG. 5.—WELL SPACING

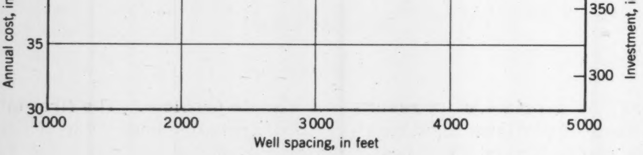


FIG. 6.—ECONOMIC SPACING OF WELLS

***Time.*—The approximate times required for a system of wells, within the scope of the program as prepared can be summarized. For the Initial run the Preparation involved 15 min for input data sheets, 5 min to check computer**

sub routines, 3 min to load program tape, 3 min to load input data, and 4 min for the computer accuracy check. The operation, involving computing and print-out, took 6 min, causing a total time for the initial run of 36 min. The supplementary runs involved 2 min to prepare and load data, and 6 min for the operation (computing and print-out) producing a total time of 8 min.

CAPACITY OF A WELL SYSTEM

Program.—Capacity of a well system involves the capacity of a number of wells in any geometric configuration including existing pumping wells, standby well, future wells and recharge wells. The objective of this program is to provide the hydrologist with a rapid method for computing the collective capacity of a group of individual wells. This is a problem frequently encountered in both new and old well systems. Basically, a need for the use of this program would develop in the solution for one or more of the following situations affecting net well field capacity:

1. The interference of each pumping well upon every other well in a system.
2. The effect of starting and stopping selected wells.
3. The effect of drilling additional wells for expansion. Expected capacity of new wells and their effect on capacity of existing wells.
4. The potential capacity improvement with artificial recharge.
5. The overall effect of improving specific capacity by chemical cleaning.
6. The overall effect of significant changes in static water level.

As in previous problems, computations are based upon the Theis equation, however, in programming this type of problem it was recognized that all computations would involve relatively large values of t. Accordingly the following modified formula was used:

$$s = \frac{114.6\,Q}{T}\left[\ln\frac{T\,t}{1.87\,r^2 S} - 0.577216\right] \quad \ldots\ldots\ldots \quad (7)$$

Since solution of this problem involves use of many different values of r as the interference effect of various wells is computed in sequence, the equation has been re-written in the form

$$s_1 = \frac{114.6\,Q}{T}\left[\ln T + \ln\frac{t}{1.87\,S} - 0.577216\right] - \frac{114.6\,Q}{T}\ln r^2 \quad \ldots \quad (8a)$$

or

$$s_1 = A\,B - A\ln r^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8b)$$

the form of which is the adaptation used for the computer.

This program computes the amount of water than can be pumped from a system of irregularly spaced wells for any specified drawdown. It has a capacity for wells in any configuration of layout up to ten wells. Pumping wells, recharge wells, and standby wells can be included. It is very easy to vary the number of wells for which computation is to be made, and to vary their pumping or recharge rates. The program will compute the time of pumping required to lower water levels to the specified allowable drawdown and will compute the radius of influence. If desired, a definite time can be specified for the computations.

The program begins by computing the interference that each well produces when pumped at a specified rate on every other well. When all interferences have been computed, the computer adds up the total interference for each well. This, together with the drawdown due to specific capacity, is then compared with the total allowable drawdown specified. A capacity pumping rate is then computed for each well by adjusting the total drawdown to the total allowable drawdown. This is the optimum pumping rate and the computer makes a new series of computations for each well using this optimum rate. Following this, the goal-capacity specified for the system is prorated to each well to take its optimum percentage of the goal capacity. The rate of flow and water level elevations are then computed for operation at this capacity.

The complete procedure as outlined is accomplished by essentially repeating the same cycle of computations three times. These computations cover performance at (1) precent capacity, (2) optimum capacity and (3) goal capacity. At the conclusion of each, a separate print-out is made. Between each series of computations a recomputation of Q is made and printed. At the end of the third series, the computer will automatically stop and reset itself to receive new data.

It will be recognized that each series of computations actually represents a problem condition with new data. Obviously some situations do not require answers to all three of these conditions. Provision has been made, therefore, to elect one, two or all three series of computations as desired.

Computation Sequence.—The following is a simplified sequence of procedure as set up for the computer:

Calc "t" and "R" (If not specified)

Calc A and B in equation for "s_1"

Calc values of "s_1" for each well at each "r"

Retain all values of "s_1" in working storage

Add values of s_1 to get Σs_1 for each well

Adjust each well rate to optimum

Compute total drawdown and elevation of water level

Print-Out

Modify: -

First Series - Replace "Q" with optimum - Repeat comp.

Second Series - Allocate goal "Q" to each well - Repeat comp.

Third Series - Stop computation

Print-Out.—Fig. 7 is a sample print-out obtained from this program for the series of computations only. As previously mentioned, three series of computations may be made. Each series will result in a print-out similar to the one shown. As before, the first tabulation is a print-out of pertinent input data. This is followed with a tabulation of the data for each well for which a computation has been made. This includes the well number, limiting s or total allowable drawdown, either the specified or computed rate of flow, the computed total interference indicated as Σsi, the total computed drawdown, and lastly the final computed elevation of the water level in each well.

In Fig. 7 the rate of flow for each well has been specified and the resulting water levels computed. In Fig. 8 the computer has altered the rate of flow of each well so that all wells are drawn-down approximately to the limiting s of 106 ft. The static level in the problem is at elevation -15 ft. Thus, the water level in each well is about at elevation -121. In Figs. 9 and 10 the same conditions as in Figs. 7 and 8 are computed, but with well No. 100 added as a recharge well at a fixed rate of 300 gpm. Note that this has a significant effect on water levels in the first series of computations (Fig. 9) and a significant effect on well capacity (gallons per minute) in the second series of computations (Fig. 10).

Time.—This program requires the following approximate times based upon a system of five wells for which all three series of computations are to be completed. The initial run involves preparation which requires 30 min for input data sheets, 5 min to check computer subroutines, 3 min to load program tape, 5 min to load input data, and 8 min for the computer accuracy check. The operation of computing and print-out involves 10 min for the first series, 10 min for the second series, and 10 min for the third series. Thus, the total time for the initial run is 81 min. Supplementary runs involve 3 min to prepare and load data, and 30 min for operation (10 min for each series,) for a total time of 33 min.

CONCLUSIONS

These accomplishments are but a small beginning in terms of the mathematical and technical potential for the application of computers in this field. With computers to handle the mathematical details, two significant results are probable. Specialists in ground water hydrology will be capable of more work of greater scope, including research in the application of mathematics. Others, specifically engineers with only a limited knowledge of ground water hydrology, will be able to utilize these prepared mathematical procedures as required to do better work faster.

A great deal of the cost involved in using computers is the cost of the technical time required for adapting the mathematics for use with a computer, planning the flow diagram, preparing the detailed program sheets, typing the program tape, and debugging the program to put it in perfect operating condi-

```
                WELL CAPACITIES
                T                  .2000000  05
                S                  .2000000  03-
                Tot GPM            .1200000  04
                Lmtg "s"           .1060000  03
                DAYS "t"           .1825000  04
                CONE R             .2340000  06

WELL NO         Limtg"s"           GPM               Σsi              Tot DD           EL WL
.7000000  01    .1060000  03       .3000000  03      .5482275  02     .7982275  02     .9482275-  02
.6000000  01    .1060000  03       .3000000  03      .5521810  02     .8021810  02     .9521811-  02
.4000000  01    .1060000  03       .3000000  03      .5642829  02     .8142829  02     .9642830-  02
.3000000  01    .1060000  03       .3000000  03      .5295722  02     .7795722  02     .9295722-  02
```

FIG. 7.—CAPACITY OF A WELL SYSTEM—FIRST SERIES OF COMPUTATIONS

```
                WELL CAPACITIES
                T                  .2000000  05
                S                  .2000000  03-
                Tot GPM            .1593245  04
                Lmtg "s"           .1060000  03
                DAYS "t"           .1825000  04
                CONE R             .2340000  06

WELL NO         Limtg"s"           GPM               Σsi              Tot DD           EL WL
.7000000  01    .1060000  03       .3983826  03      .7272193  02     .1059205  03     .1209205-  03
.6000000  01    .1060000  03       .3964192  03      .7337061  02     .1064055  03     .1214056-  03
.4000000  01    .1060000  03       .3905276  03      .7543666  02     .1079806  03     .1229806-  03
.3000000  01    .1060000  03       .4079160  03      .6970033  02     .1036933  03     .1186933-  03
```

FIG. 8.—CAPACITY OF A WELL SYSTEM—SECOND SERIES OF COMPUTATIONS

```
                WELL CAPACITIES
                T                  .2000000  05
                S                  .2000000  03-
                Tot GPM            .1200000  04
                Lmtg "s"           .1060000  03
                DAYS "t"           .1825000  04
                CONE R             .2340000  06

WELL NO         Limtg"s"           GPM               Σsi              Tot DD           EL WL
.7000000  01    .1271396   03      .3000000  03      .5482275  02     .7982275  02     .7368310-  02
.6000000  01    .1262909   03      .3000000  03      .5521810  02     .8021810  02     .7492717-  02
.4000000  01    .1261334   03      .3000000  03      .5642829  02     .8142829  02     .7629485-  02
.3000000  01    .1236743   03      .3000000  03      .5295722  02     .7795722  02     .7528293-  02
.1000000  03    .1000000-  03      .3000000- 03      .7923833  02     .5423832  02     .6923833-  02
```

FIG. 9.—CAPACITY OF A WELL SYSTEM—FIRST SERIES—WITH RECHARGE WELL

```
                WELL CAPACITIES
                T
                S
                Tot GPM
                Lmtg "s"
                DAYS "t"
                CONE R

WELL NO         Limtg"s"           GPM                Σsi              Tot DD           EL WL
.7000000  01    .1271396   03      .4778323   03      .8604946  02     .1258688  03     .1197292-  03
.6000000  01    .1262909   03      .4723034   03      .8703225  02     .1263909  03     .1210999-  03
.4000000  01    .1261334   03      .4647037   03      .8941100  02     .1281363  03     .1230029-  03
.3000000  01    .1236743   03      .4759313   03      .8318137  02     .1228423  03     .1201680-  03
.1000000  03    .1000000-  03      .3000000-  03      .1248418  03     .9984176  02     .1148418-  03
```

FIG. 10.—CAPACITY OF A WELL SYSTEM—SECOND SERIES—WITH RECHARGE WELL

tion. Once this has been completed, significant savings in time and cost begin to be realized as opportunities develop for reuse of the program on repeated problems.

SUMMARY

Four problems have been programmed for use on a general purpose computer. In addition, the problem of determining aquifer constants from test data has been programmed on the Univac. The economics of this type of work dictate the use of a computer only when the scope of the job is of such magnitude that considerable time and engineering cost can be saved over previously used testing, graphical, and manual computing methods.

In such cases, a decision is needed as to what type and size of computer is justified. It would appear that the first two types of problems are more adaptable to a digital computer, whereas the latter two are more suitable to an analog. While this may be technically correct, the most important consideration is to provide flexible programs for these hydrologic problems which can be repeatedly used on a single readily available computer with minimum effort and at minimum cost. A general purpose digital computer is a practical selection.

ACKNOWLEDGMENTS

The computer procedures described herein were devised at the Engineering Service Division of the E. I. DuPont de Nemours Co., of Wilmington, Delaware.

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2. Wisler, C. O. and E. F. Brater. "Hydrology," 1949.
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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3142

DRAWDOWN DUE TO PUMPING FROM AN UNCONFINED AQUIFER

By Robert E. Glover,[1] F. ASCE, and Morton W. Bittinger[2]

SYNOPSIS

First approximation treatments of the drawdown pattern about a pumped well in an unconfined aquifer are commonly based upon the simplifying assumption that the drawdown does not alter the transmissibility of the aquifer. A second approximation is developed in this paper which accounts for the reduction of the transmissibility by the drawdown. The first approximate solution can be plotted as a single line on a graph having the dimensionless parameters $y/[Q/(2\pi KD)]$ and $r/\sqrt{4\alpha t}$ as ordinate and abscissa. The improved solution plots as a series of lines on such a graph. These lines are indentified by the parameter $(Q/2\pi KD^2)$. The method of using the graph is the same as before but it is now possible to add a family of curves indicating the ratio of the drawdown to the original depth.

INTRODUCTION

A basic assumption in the derivation of commonly used non-equilibrium well formulas is that the transmissibility remains constant with time. This assumption is met in confined aquifers if they are not dewatered. However, it is impossible to completely satisfy the assumption of constant transmissibility in unconfined situations. For such situations the transmissibility decreases as the drawdown of the water table about the well reduces the area of flow. In thick aquifers, where the drawdown is small compared to the original thickness of

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Note.—Published essentially as printed here, in September, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2594. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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the saturated material, the assumption is not seriously violated and the currently used formulas provide a good approximation of the actual conditions. But in situations where the drawdown is large compared to the original saturated thickness the present well formulas become inaccurate.

This paper develops a second approximation equation which takes into consideration the reduction in saturated thickness about a water table well. In addition, a method of refining this approximation is outlined. Like formulas already in use, the one presented here conforms to the Dupuit-Forschheimer concepts and should give valid results for computation of the free surface over that area about a well in which the slope of the water table is small.

Notation.—The letter symbols adopted for use in this paper are defined and arranged alphabetically, for convenience of reference, in Appendix II.

FIRST APPROXIMATION

The flow through a cylindrical surface of height D-y, at distance r from the well, is:

$$F = -2\pi r (D-y) K \frac{\partial y}{\partial r} \dots\dots\dots\dots\dots (1)$$

The change in flow with respect to the radius r, is:

$$\frac{\partial F}{\partial r} = -2\pi K \frac{\partial}{\partial r}\left[(D-y)\, r \frac{\partial y}{\partial r}\right] \dots\dots\dots\dots\dots (2)$$

Another expression for F may be written in terms of the change in storage of water beyond the radius r, with respect to time, as follows:

$$F = \int_r^\infty 2\pi r\, dr\, V \frac{\partial y}{\partial t} \dots\dots\dots\dots\dots (3)$$

Or in terms of the change of flow with respect to the radius, Eq. 3 may be written as:

$$\frac{\partial F}{\partial r} = 2\pi r V \frac{\partial y}{\partial t} \dots\dots\dots\dots\dots (4)$$

By combining Eqs. 2 and 4, simplifying and rearranging, the following nonlinear differential equation is obtained:

$$\alpha (D-y)\left(\frac{\partial^2 y}{\partial r^2} + \frac{1}{r}\frac{\partial y}{\partial r}\right) - \alpha \left(\frac{\partial y}{\partial r}\right)^2 = D \frac{\partial y}{\partial t} \dots\dots\dots\dots (5)$$

Because of the difficulties in solving Eq. 5, the more common development is based on the assumption that flow occurs through the entire depth (D) of the aquifer, or that y is negligible in comparison to D. Thus, starting again with Eq. 1, but neglecting y, in the sum (D-y) and following similar steps, a linear partial differential equation may be developed. This is:

$$\alpha\left(\frac{\partial^2 y}{\partial r^2} + \frac{1}{r}\frac{\partial y}{\partial r}\right) = \frac{\partial y}{\partial t} \dots\dots\dots\dots\dots (6)$$

If the variable,

$$u = \frac{r}{\sqrt{4\,\alpha\,t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

is substituted into Eqs. 5 and 6, they will be reduced to ordinary differential equations in y and u. In this manner, Eq. 6 may be reduced to:

$$\frac{\partial^2 y}{\partial u^2} + \left(\frac{1}{u} + 2\,u\right)\frac{\partial y}{\partial u} = O \quad \ldots\ldots\ldots\ldots\ldots (8)$$

If

$$p = \frac{\partial y}{\partial u} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Eq. 8 is reduced to the following first order differential equation:

$$\frac{dp}{du} + \left(\frac{1}{u} + 2\,u\right)p = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

An integrating factor for Eq. 10 is, $u\,e^{u^2}$, thus, giving the solution:

$$p\,u\,e^{u^2} = C_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

A particular solution, subject to the conditions that the aquifer is of infinite lateral extent, the discharge of the well is Q, and that y = 0 for all values of r when t = 0, is:

$$y = \frac{Q}{2\,\pi\,K\,D}\int_u^\infty \frac{e^{-\beta^2}}{\beta}\,d\beta \quad \ldots\ldots\ldots\ldots\ldots (12)$$

This is a form of the exponential integral, since,

$$\int_u^\infty \frac{e^{-\beta^2}}{\beta}\,d\beta = \frac{1}{2}\int_{u^2}^\infty \frac{e^{-\upsilon}}{\upsilon}\,d\upsilon = -\frac{1}{2}E\,i\left(-u^2\right) \quad \ldots\ldots\ldots (13)$$

Values for each have been tabulated.[3, 4, 5] A well pumping formula expressed in terms of the exponential integral has been described by Thesis.[6]

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[3] "Heat Conduction." by L. R. Ingersoll, O. J. Zobel, and A. C. Ingersoll, McGraw-Hill Book Co., Inc., New York, 1948. (A tabulation of the function $\int_x^\infty \frac{e^{-\beta^2}}{\beta}\,d\beta$ is given in Appendix F.)

[4] "Tables of Functions with Formulas and Curves," by E. Jahnke and F. Emde, Dover, 1945.

[5] Federal Works Agency, Work Prof. Admin. for the City of New York, Table of Sine, Cosine and Exponential Integrals, Vol. 1 and 2, Tables MT5 and MT6. Superintendent of Documents, Washington, D. C., 1940.

[6] "The Relation Between the Lowering of the Piezometric Surface and the Rate and Duration of Discharge of a Well using Ground Water Storage," by C. V. Theis, Transactions, AGU, Vol. 16, 1935, p. 519.

Eq. 12 constitutes the first approximation for the unconfined situation. In developing a second approximation which takes into consideration the effect of drawdown it can be anticipated that the ratio of drawdown to the original saturated thickness (y/D) will be involved.

SECOND APPROXIMATION

An iteration procedure, based upon simple physical concepts, provides an effective means for obtaining the second and successive improved approximations. Beginning again with the basic Eqs. 1 and 3, the expressions to be used in the iteration procedure can be developed.

By rearranging Eq. 1 and substituting the variable $u = \frac{r}{\sqrt{4\,\alpha\, t}}$ it can be put in the form:

$$-2(D-y)\,\frac{\partial y}{\partial u} = \frac{F}{\pi\, K\, u} \quad \ldots\ldots\ldots\ldots\ldots (14)$$

Integration of Eq. 14, subject to the requirement that y should approach zero as u approaches infinity yields the following expression:

$$(D-y)^2 = D^2 - \int_{u_1}^{\infty} \frac{F}{\pi\, K}\,\frac{du}{u} \quad \ldots\ldots\ldots\ldots\ldots (15)$$

By letting

$$\sigma = \frac{Q}{2\,\pi\, K\, D^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

and substituting this relation into Eq. 15, an expression for y/D may be obtained in the form:

$$\frac{y}{D} = 1 - \sqrt{1 - 2\,\sigma \int_{u_1}^{\infty} \frac{F}{Q}\,\frac{du}{u}} \quad \ldots\ldots\ldots\ldots\ldots (17a)$$

Another useful relationship can be developed by using the substitution:

$$\frac{y}{D} = \frac{\sigma\, y}{\left(\frac{Q}{2\,\pi\, K\, D}\right)} = \sigma\,\psi \quad \ldots\ldots\ldots\ldots\ldots (17b)$$

Then Eq. 17a takes the form:

$$\psi = \frac{1}{\sigma}\left(1 - \sqrt{1 - 2\,\sigma \int_{u_1}^{\infty} \frac{F}{Q}\,\frac{du}{u}}\right) \ldots\ldots\ldots\ldots\ldots (18)$$

Similarly, substituting the variable u into Eq. 3, puts it in the form:

$$F = -\int_{u_1}^{\infty} 4\,\pi\, K\, D\,\frac{\partial y}{\partial u}\, u^2\, du \quad \ldots\ldots\ldots\ldots\ldots (19)$$

And since

$$\psi = \frac{y}{\left(\frac{Q}{2\,\pi\,K\,D}\right)} = \frac{\left(\frac{y}{D}\right)}{\left(\frac{Q}{2\,\pi\,K\,D^2}\right)} \quad \ldots\ldots\ldots\ldots\ldots (20)$$

Eq. 19 may be rewritten as follows:

$$\frac{F}{Q} = -\,2 \int_{u_1}^{\infty} u^2\,\frac{\partial\psi}{\partial u}\,du \quad \ldots\ldots\ldots\ldots\ldots (21)$$

These considerations yield Eqs. 18 and 21, relating ψ and F/Q to u. By alternately solving each, using values previously obtained from the other, increasingly improved values of ψ can be obtained. The integrals in Eqs. 18 and 21 are conveniently evaluated by graphical integration procedures. In working out selected values, ψ was found to converge rapidly.

TABLE 1.—COMPARISON OF THE FIRST, SECOND AND THIRD APPROXIMATIONS OF ψ WHEN $\sigma = 0.1$

u	ψ			y/D
	1st approximation	2nd approximation	3rd approximation	
(1)	(2)	(3)	(4)	(5)
0.00482	---	---	10.00	1.000
0.00500	5.01	10.00	9.16	0.916
0.00600	4.83	8.14	7.90	0.790
0.00700	4.67	7.44	7.28	0.728
0.00800	4.54	6.97	6.82	0.682
0.00900	4.42	6.60	6.46	0.646
0.01000	4.32	6.30	6.18	0.618
0.10000	2.02	2.28	2.23	0.223

An excellent starting point for the iteration procedure can be obtained by substituting a value of $\frac{\partial y}{\partial r}$ obtained from the first approximation Eq. 12 into Eq. 1 with the quantity (d - y) replaced by D. This yields

$$\frac{F}{Q} = e^{-u^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

A substitution of Eq. 22 into Eq. 18 then gives a second approximation for ψ of the form:

$$\psi = \frac{1}{\sigma}\left(1 - \sqrt{1 - 2\,\sigma \int_{u_1}^{\infty} \frac{e^{-u^2}}{u}\,du}\right) \quad \ldots\ldots\ldots\ldots (23)$$

A comparison of values for the 1st, 2nd and 3rd approximation of ψ when σ = 0.1 is given in Table 1. The third approximation was obtained by graphical integration based on Eqs. 18 and 21. For most field conditions the second approximation, as computed from Eq. 23, is usually sufficiently accurate.

It will be noted that the third approximation drawdowns are slightly reduced as compared to those of the second approximation. The use of the second approximation, therefore, is somewhat on the safe side because actual drawdowns should be less than those estimated.

Fig. 1 has been plotted from Eq. 23. The lower line represents the first approximation values of ψ , whereas the other curves represent the deviation from the first approximation for several values of σ. The major deviation is found when y/D is larger than 0.5. Eq. 23 provides a simple means for constructing the large scale charts needed for every day use.

EXAMPLE

Data from a pumping test conducted near Mosca, Colorado in 1954 are used in the following example to illustrate the use of Fig. 1 to determine aquifer characteristics. This test was previously reported by Zee, Peterson and Bock.[7] The pertinent date are Q = 167 gpm = 0.37 cfs, D = 22 ft, r = 25.8 ft and are given in Table 2.

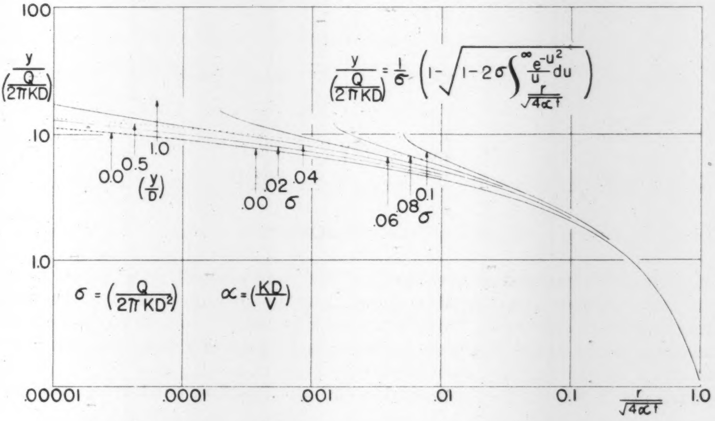

FIG. 1.—SECOND APPROXIMATION OF DOWNDRAW

Procedure.—Plot y versus $r/\sqrt{t}$ on transparent log graph paper of the same scale as the master chart (Fig.1). Match the resulting curve to the proper curve of the master chart for the situation of y/D = 0.14 to 0.19, then determine the point on the master chart over which an index point on the plotted graph lies. For convenience the index point y = 1.0, $(r/\sqrt{t})$ = 1.0 was chosen.

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[7] "Flow into a Well by Electric and Membrane Analogy," by Chang-Hung Zee, Dean F. Peterson, and Robert O. Bock, Transactions, ASCE, Vol. 122, 1957, p. 1088.

When the curves were matched, this index fell on the point

$$\frac{y}{\left(\frac{Q}{2\,\pi\,K\,D}\right)} = 0.77 \qquad \frac{r}{\sqrt{4\,\alpha\,t}} = 0.68$$

Thus:

$$y\colon 1.0 = \frac{y}{\left(\frac{Q}{2\,\pi\,K\,D}\right)} : 0.77$$

and

$$K = 0.00206 \text{ ft per sec}$$

This compares with values of 2.23×10^{-3} and 2.28×10^{-3} ft per sec obtained from the original analyses.

TABLE 2.—PERTINENT DATA

t, in sec (1)	y, in ft (2)	$\frac{r}{\sqrt{t}}$ (3)	y/D (4)
56,280	3.17	0.1088	0.14
70,680	3.36	0.0970	0.15
87,480	3.49	0.0872	0.16
148,680	3.91	0.0669	0.18
174,480	4.05	0.0618	0.18
232,680	4.24	0.0535	0.19

CONCLUSIONS

A second approximation treatment of the case presented by a well drawing water from an unconfined aquifer, based upon the Dupuit-Forschheimer idealization, has been obtained. This improved treatment accounts for the effect of drawdown on the areas available for the flow of ground water. The ratio of the drawdown to the original saturated thickness of the aquifer (y/D) therefore appears explicitely in the improved solution. The flow conditions are specified by a second parameter of the form $\frac{Q}{2\,\pi\,K\,D^2}$. This parameter is interpretable as the ratio of the well discharge Q to the flow which would pass through a cylindrical surface of height D and radius D under the action of a unit gradient. The ratio $\frac{Q}{2\,\pi\,K\,D^2}$ therefore relates the pumping rate to a measure of the aquifer capacity. An iteration procedure is described by which the second approximation can be further refined if desired.

APPENDIX I.—ADDITIONAL REFERENCE

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1. Boulton, N.S. "The Drawdown of the Water-Table under Non-Steady Conditions near a pumped Well in an Unconfined Formation," Proceedings of

the Institution of Civil Engineers, V3, Pt. III. No. 2, August 1954, p. 564.

APPENDIX II.—NOTATION

The following symbols are adopted for use in the paper. The dimensional characteristics are indicated in parenthesis. The terms L and T represent length and time respectively.

D = original saturated depth of the unconfined aquifer, (L);

e = 2.61828 +;

F = flow through a cylindrical surface of radius r and height $(D-y)$, (L^3/T);

K = permeability of the aquifer, (L/T);

p = A derivative;

Q = flow of the well, here considered to be constant, (L^3/T);

r = radius drawn from the center of the well, (L);

t = time, (T);

$u = \dfrac{r}{\sqrt{4\,\alpha t}}$ (Dimensionless);

V = drainable voids in the aquifer expressed as a ratio to the gross volume, (Dimensionless);

y = drawdown of the water table caused by pumping the well, (L);

$\alpha = \dfrac{K\,D}{V}$, (L^2/T);

π = 3.14159+;

$\sigma = \dfrac{Q}{2\,\pi\,K\,D^2}$; and

$\psi = \dfrac{y}{\dfrac{Q}{2\,\pi\,K\,D}}$.

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TRANSACTIONS

Paper No. 3143

IRRIGATION AND DRAINAGE POTENTIALS IN HUMID AREAS

Marion Clifford Boyer,[1] F. ASCE

SYNOPSIS

In the words of the demographers, the world's population is "exploding." If present growth rates continue to the year 2000, the population of the United States will have increased approximately 1.82 times, Communist China 2.16 times, and other areas of the world by like amounts. It is estimated that by the year 2000 the United States will be producing approximately one-third more food than needed for its own people, while Communist China and much of the rest of the world will be producing less than one-half the food needed. Under those circumstances mass starvation would be inevitable. Civil engineers must accept the responsibility for making the cultivated lands yield most efficiently through proper irrigation and drainage, particularly in the humid areas of the world.

INTRODUCTION

The conquest of bacteria and viruses has so reduced the incidence of disease throughout the world that populations long stagnant because of the tragic balance between deaths and births are suddenly increasing at prodigious rates. Unless Man soon gains control over his own fecundity, the world's population will be reduced to the simple task of toiling only to produce its food and shelter. Food supplies in many areas of the earth have never been adequate for the populations which they sustain. Should this population expansion continue, starvation will reduce the multitudes. Engineers should be aware of this problem as they are in position to contribute to the more efficient production of food

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Note.—Published essentially as printed here, in September, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2593. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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and fiber. They must accept the responsibility of evaluating the situation and doing whatever is within their power to alleviate it, through increased food production.

Every piece of land on the earth's surface which is capable of being cultivated may one day be called upon to yield most efficiently its share of food and fiber for Man's use and for the sustenance domestic animals. Efficient use of

TABLE 1.—POPULATIONS AND FOOD SUPPLIES OF COUNTRIES OF THE WORLD, 1937 AND 1957

Region	Land area, in sq miles	Population characteristics				Food supply, 1957, in kilocalories	
		Total, millions per sq mile			Increase, in percentage per year	Per person	Per sq mile
		1937	1957	1957			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
United States							
Continental	3,022,387	128.9	171.2	56.8	1.43	3.10	177
Alaska	586,400	.07	.21	.4	6.88	3.50[e]	1.4
Hawaii	6,423	.40	.61	9.5	2.05	3.00[e]	28.5
Canada	3,851,116	11.34	16.59	4.3	1.92	3.14	13.5
Central and So. America							
Caribbean	1,824,338	45.87	74.18	40.7	2.41	2.50[c]	102
Argentina	1,072,748	13.49	19.87	18.6	1.95	2.98	55.5
Brazil	3,287,204	38.69	61.27	18.6	2.32	2.52	47.0
Other countries	1,540,508	19.56	28,52	18.5	1.92	2.80[c]	51.8
Soviet Union	8,649,821	170.47[a]	200.20[b]	23.2[b]	1.04	2.50[c]	58.0
Australia & New Zealand	3,078,056	8.43	11.88	3.9	1.73	3.30	12.5
Scandinavia	428,808	12.83	15.20	35.5	.83	3.10[c]	110
Central Europe	1,367,866	339.8	378.4	276	.52	2.72	750
Africa							
Egypt (United Arab Rep)	386,101	16.01	24.03	62.3	2.05	2.59	162
Union of So. Africa	472,359	9.80	14.17	30.0	1.84	2.65	79.5
Other countries	10,642,000	- -	186	17.5	2.05[c]	2.50[c]	43.8
Near East	1,523,800	57.33	79.41	52.3	1.66	2.50[c]	131
Far East							
India	1,267,094	304.3	392.4	309	1.29	1.89	584
Pakistan	364,797	66.01	84.45	232	1.24	2.00	464
Japan	142,726	70.04	90.90	637	1.32	2.07	1,310
Philippine Islands	115,600	15.44	22.69	196	1.95	1.94	380
Other countries	1,345,714	155.66	197.89	147	1.21	2.00[c]	294
Communist China	3,768,736	446.9	640.0	170	1.81	1.90[c]	323

[a] Data for the year 1939. [b] Data for the year 1956. [c] Estimated.

the soil requires that it be irrigated and drained. It is this potential for irrigation and drainage of much of the earth's surface, the humid areas in particular, which offers the greatest challenge to the civil and the agricultural engineer.

PRESENT WORLD POPULATION AND FOOD SUPPLY

Much has been written since the days of Malthus concerning the balance between the world's population and its food supply. Pessimists have decried the fact that within a very few years starvation will be the rule. Optimists have pointed to the vast land areas not now cultivated and to the depths of the sea as sources of food which will supply Man's needs, no matter how prodigiously the human population expands. It is certain that neither view is entirely correct. A dispassionate examination of the facts concerning populations and food supplies of the nations of the world will help to clarify the picture and to show the urgent need for increasing the crop yields of the earth's soils.

Table 1 shows the present population characteristics for the major regions of the world and for the principal nations in each region. The data[2] include land area, total populations in 1937 and 1957, and food supply.

GROWTH OF THE WORLD POPULATION

One can only conjecture as to the probable growth of the world's population in future years. Several methods of estimating population growth have been suggested. The writer has chosen to use the exponential formula which, in the form of "compound interest," is given by

$$P = P_0 (1 + i)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Eq. 1 has been used to determine the rates of population growth i in percentage per year for twenty years prior to 1957, utilizing data contained in Table 1, and to extrapolate forward to the year 2000.

The figures of population for 1937 and 1957 were substituted in Eq. 1 as P_0 and P, respectively, and the coefficient i determined. For the extension, the regions covered in Table 1 were combined into larger areas, Table 2, the regional coefficient i determined, and Eq. 1 again solved to extend the increase forty-three years from 1957 to the year 2000 for these larger areas. These data are given in Table 3.

INCREASE OF THE WORLD'S FOOD SUPPLY

There are many nations on the earth today whose people are continuously on the verge of starvation. As the population explosion continues, if the production of food does not keep pace, mass starvation is inevitable. If production does not expand at a greater rate, the fringe nations can never hope for that raised standard of living which is the goal of all the earth's people.

A world-wide tabulation of the estimated net food supply, in calories per day per person, available for human consumption is presented elsewhere.[3] These data have been included in Tables 1 and 2. The product of the available food supply and the population density in persons per square mile gives a figure of food supply as a measure of land yield, in calories per square mile per day (tabulated as kilocalories).

The probable food supply by the year 2000, available and required, is even more difficult to forecast than population growth. This has been attempted by starting with an estimate of the daily requirement per person at the year 2000, based on the 1957 United States food consumption. This figure was slightly reduced for the year 2000 estimate, to compensate for the present surplus production. The per person requirement for the other regions of the World was then estimated, using the United States figure, but making it larger for the cooler regions and smaller for the warmer, in approximately the same proportion as at present.

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[2] Statistical Abstracts of the United States, U.S. Dept. of Commerce, Govt. Printing Office, 1956 and 1959, Section 1.

[3] Ibid., 1959, Table 1213, p. 929.

TABLE 2.—POPULATIONS AND FOOD SUPPLIES BY REGIONS OF THE WORLD, 1957

Region	Population characteristics				Food supply, in kilocalories per day	
	Land area, in sq miles	Total, 1957, in millions	Per sq mile	Increase, percentage per year	Per person	Per sq mile
(1)	(2)	(3)	(4)	(5)	(6)	(7)
United States, Continental	3,022,387	171.2	56.8	1.43	3.10	177
Canada	3,851,116	16.59	4.3	1.92	3.14	13.5
Central & So. America	7,724,793	183.8	23.8	2.25	2.50[a]	59.5
Soviet Union	8,649,821	200.2	23.2	1.00	2.50[a]	58.0
Australia & New Zealand	3,078,056	11.88	3.9	1.73	3.30	12.9
Scandinavia & Cent. Europe	1,796,674	393.6	219	.57	2.85	625
Africa	11,500,000	224.2	19.5	2.05[a]	2.50[a]	48.8
Near East	1,523,800	79.41	52.3	1.66	2.50[a]	131
Far East	3,235,931	788.3	244	1.28	1.95[a]	476
Communist China	3,768,736	640.0	170	1.81	1.90[a]	323

[a] Estimated on basis of data for some countries in the region.

RELATION BETWEEN POPULATION GROWTH AND FOOD SUPPLY

The data contained in Tables 2 and 3 are presented graphically as bar diagrams for each region in Fig. 1. The left bar in each instance is the population in 1957 and that as estimated for the year 2000, plotted with the number per square mile as the ordinate on the land area as a base. The area of this diagram is the population volume. The right bar is the food supply, in kilocalories per square mile, over the land area as a base. On this bar the figure in parentheses is the estimated food supply available at the year 2000, the other figure is the estimated supply required in that year. If the required supply is less than that available, the difference represents a surplus. This is shown on the diagram by a dotted area. If, on the other hand, the required supply is greater than that available, the supply is deficient and the region cannot feed its people from its own crops. This deficiency is shown by the heavy-barred areas. It is evident that some regions of the world are in grave danger of soon reaching that sad state—mass starvation—foreseen one hundred sixty years ago by Malthus, because they cannot produce sufficient food for their people.

FARM LAND IN THE UNITED STATES

The United States of America, by reason of the great technological skills of its people, is an industrial giant among nations. Technological developments in agriculture have made possible such a yield of food and fiber that the country has a great surplus each year. Many of the nations of the world would welcome this condition. That they are not so favored is due in part to the densities of their populations and in part to primitive methods of land cultivation.

The climate in the United States ranges from arid to humid and irrigation and drainage practices have been tailored to fit the climate of each region. On the arid lands of the West crops can be grown only when water is supplied by irrigation. Drainage consists in part in preventing the saturation of low spots by water escaping below the root zone of the cropped land and collecting in pockets, and in part in carrying off that part of the irrigation water which escapes beyond the cropped land by overland flow or seepage to drains.

Across the Midcontinent the climate ranges from semi-arid to sub-humid. The need for irrigation water becomes smaller and for drainage greater.

TABLE 3.—POPULATIONS AND FOOD SUPPLIES BY REGIONS OF THE WORLD, ESTIMATED FOR THE YEAR 2000

Region	Population forecasts			Food supply, in kilocalories per day per sq mile			
	Land area, in sq miles	Computed ratio 2000/1957	Total in year 2000, in millions	Per sq mile	Per person (estimated)	Required	Maximum available
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
United States, Continental	3,022,387	1.82	311	103	3.0	309	400
Canada	3,851,116	2.27	38	9.8	3.2	32	50
Central & So. America	7,724,798	2.61	478	62	2.8	174	300
Soviet Union	8,649,821	1.55	300	35	2.8	98	150
Australia & New Zealand	3,078,056	2.09	25	8	3.0	24	40
Scandinavia & Cent Europe	1,796,674	1.25	492	273	2.8	765	500
Africa	11,500,000	2.40	538	47	2.5	118	200
Near East	1,523,800	2.03	161	106	2.3	244	150
Far East	3,235,931	1.73	1,370	423	2.3	975	500
Communist China	3,768,736	2.16	1,380	366	2.5	915	400

In the South and East irrigation is used to increase the yield of crops which will produce when watered by natural precipitation alone. Drainage assumes great importance, as not only must the soil of the root zone be kept unsaturated, but storm water must rapidly be taken off the land surface as well.

For purposes of this study the continental United States has been divided into nine regions and an analysis made of the acreages farmed, irrigated, and drained in each. Data[4] are contained in Table 4.

Fig. 2 shows the selected regions, together with line diagrams showing the percentage of each framed during the years 1930, 1940, 1950, and 1954. There is a wide range in the percentage of total land area farmed throughout the United States. In 1954 it varied from 27.5% in New England to 89.7% in the Great Plains. Rocky and mountainous New England contains little land suitable

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[4] Statistical Abstracts of the United States, U.S. Dept. of Commerce, Govt. Printing Office, 1956 and 1959, Sects. 23 & 24.

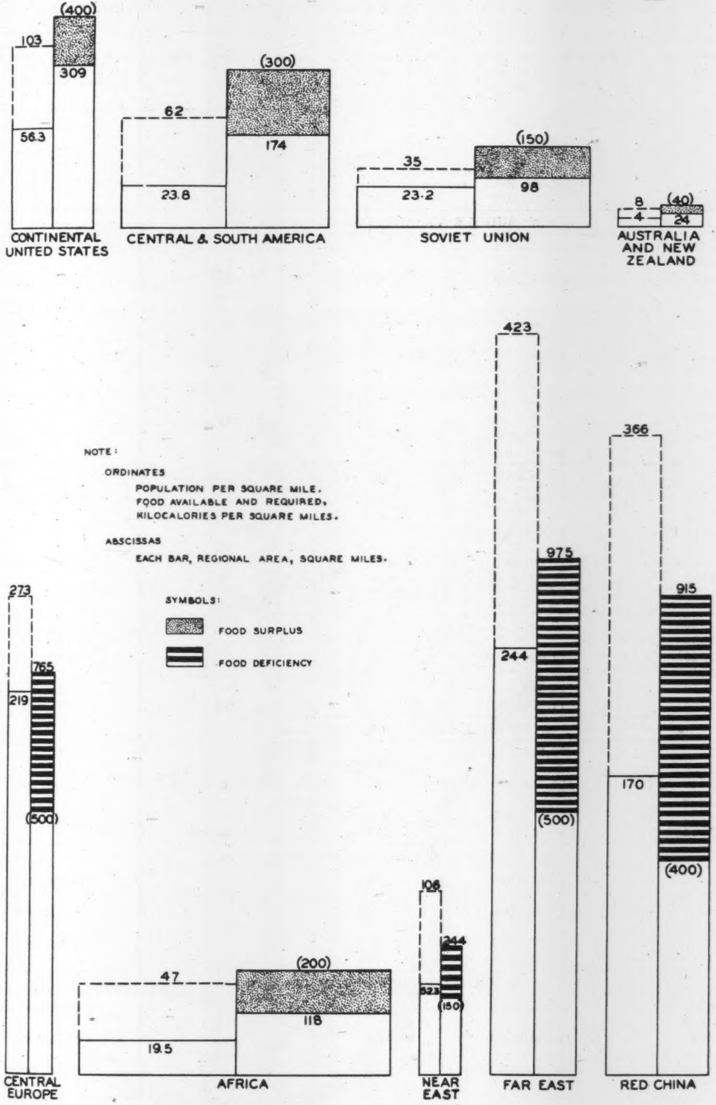

FIG. 1.—POPULATION ESTIMATES, YEARS 1957 AND 2000 AND FOOD PRODUCTION, YEAR 2000.

TABLE 4.—AREAS IRRIGATED AND DRAINED, BY REGIONS, CONTINENTAL UNITED STATES

Year (1)	Farm area, in acres (2)	Acres per farm (3)	Area irrigated, in acres (4)	Area drained, in acres (5)
(a) NEW ENGLAND (40,500,000 acres)				
1930	14,280,000	114		
1939			2,846	
1940	13,370,000	99		
1949			31,450	
1950	12,550,000	122		
1954	11,120,000	136	38,395	
(b) MIDDLE ATLANTIC (64,500,000 acres)				
1930	35,050,000	98		
1939			17,260	
1940	33,640,000	96		
1949			54,616	
1950	31,860,000	106		67,671
1954	29,900,000	116	135,886	
(c) SOUTH ATLANTIC (165,000,000 acres)				
1930	86,360,000	82		6,920,000
1939			128,037	
1940	92,560,000	83		6,810,000
1949			381,044	
1950	102,170,000	116		7,560,000
1954	98,260,000	114	536,195	
(d) EAST NORTH CENTRAL (157,000,000 acres)				
1930	110,890,000	116		33,480,000
1939			10,833	
1940	113,660,000	113		32,680,000
1949			36,237	
1950	112,100,000	126		36,020,000
1954	108,600,000	136	75,578	
(e) EAST SOUTH CENTRAL (115,000,000 acres)				
1930	72,820,000	68		4,170,000
1939			891	
1940	77,090,000	75		3,990,000
1949			6,950	
1950	79,580,000	87		4,680,000
1954	77,200,000	98	185,130	
(f) WEST NORTH CENTRAL (131,000,000 acres)				
1930	98,680,000	150		20,720,000
1939			6,286	
1940	105,490,000	157		20,240,000
1949			7,710	
1950	102,270,000	166		21,150,000
1954	100,520,000	179	44,601	
(g) WEST SOUTH CENTRAL (62,800,000 acres)				
1930	25,410,000	63		8,290,000
1939			573,381	
1940	28,040,000	76		8,810,000
1949			998,882	
1950	30,070,000	98		16,860,000
1954	29,380,000	115	1,564,557	
(h) GREAT PLAINS (408,000,000 acres)				
1930	325,310,000	282		5,980,000
1939			1,529,770	
1940	345,410,000	338		7,640,000
1949			4,291,913	
1950	363,450,000	432		9,550,000
1954	365,770,000	482	6,446,142	
(i) ROCKY MOUNTAIN AND PACIFIC (751,000,000 acres)				
1930	217,980,000	433		4,780,000
1939			15,713,626	
1940	255,590,000	503		6,200,000
1949			19,037,333	
1950	324,520,000	703		6,080,000
1954	337,430,000	797	20,959,034	

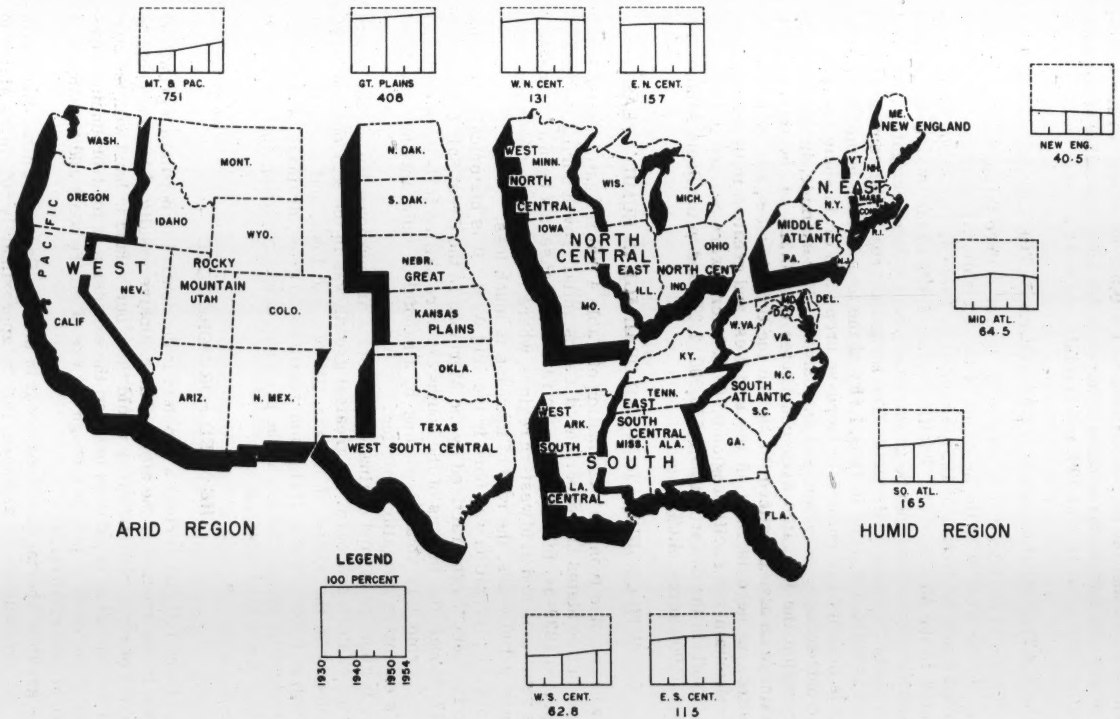
MT. & PAC.
751
GT. PLAINS
408
W. N. CENT.
131
E. N. CENT.
157
NEW ENG.
40.5
MID ATL.
64.5
SO. ATL.
165
W. S. CENT.
62.8
E. S. CENT.
115
LEGEND
100 PERCENT
1930
1940
1950
1954
ARID REGION
HUMID REGION
PACIFIC
WEST
ROCKY
MOUNTAIN
GREAT
PLAINS
WEST SOUTH CENTRAL
WEST NORTH CENTRAL
NORTH CENTRAL
EAST NORTH CENT
WEST SOUTH CENTRAL
EAST SOUTH CENTRAL
SOUTH
SOUTH ATLANTIC
MIDDLE ATLANTIC
N. EAST
NEW ENGLAND
WASH.
OREGON
CALIF
IDAHO
NEV.
MONT.
WYO.
UTAH
COLO.
ARIZ.
N. MEX.
N. DAK.
S. DAK.
NEBR.
KANSAS
OKLA.
TEXAS
MINN.
IOWA
MO.
ARK.
LA.
WIS.
MICH.
ILL.
IND.
OHIO
KY.
TENN.
MISS.
ALA.
GA.
FLA.
S.C.
N.C.
VA.
W.VA.
MD.
D.C.
DEL.
PA.
N.J.
N.Y.
VT.
NH.
ME.
MASS.
CONN.
R.I.

FIG. 2.—PERCENTAGE OF LAND UNDER CULTIVATION 1930-1954.

for farming, while the vast flatlands of the Great Plains may nearly all be cultivated. Nationwide, the cultivated area is 1,810,000 sq miles, a ratio to total area of 60%. Of this, the area actually harvested in 1954 was 528,000 sq miles, a ratio to cultivated area of 29% and to total area of 18%.

IRRIGATION AND DRAINAGE PRACTICES IN THE UNITED STATES

Irrigation is practiced to a greater or lesser degree throughout the United States. In the West it is the keystone of the agricultural economy. Without it few crops would be grown. In 1954, as noted in Table 4, 21,000,000 acres were irrigated in the Mountain and Pacific region, 2.80% of the total area. Across the Midcontinent crops can be grown most years without irrigation, but irrigation would increase their yield. The irrigated area in the semi-arid Great Plains was 6,450,000 acres in 1954, 1.58% of the total. In the humid East, irrigation is practiced principally to increase crop yield or, in some areas, to reduce frost damage. It is not used extensively. For example, in the South Atlantic region the irrigated area constitutes only 0.32% of the total in 1954.

Drainage is also practiced throughout the United States, but, conversely to irrigation, its need is least in the arid West and greatest in the humid East. In the Mountain and Pacific region 6,080,000 acres were drained in 1950, 0.8% of the total; in the Great Plains 9,550,000, 2.3%; and in the South Atlantic region 7,560,000 acres, 4.6%.

GROWTH OF IRRIGATION AND DRAINAGE IN HUMID AREAS

It will be noted from an examination of Table 4 that irrigation in the humid eastern United States has experienced a substantial growth during the fifteen-year period 1939-54. In each region the rate of increase has been very marked. In the semi-arid and arid western regions, where irrigation has been practiced upwards of 100 years, the rate of increase is much less.

American irrigation was born in the arid West. It is historically reported as having been first practiced by the Mormons in the Great Salt Lake Valley in 1847, though the Indians of the southwest were skilled irrigators centuries earlier. From this small beginning by a handful of emigrants has developed today's extensive irrigation practice.

The beginnings of land drainage in America are unknown, but probably were along the eastern seaboard. Its greatest growth was in the Midcontinent area, particularly in the states of Indiana, Illinois, and Iowa, and along the lower Mississippi River. While irrigation continues to grow, drainage shows comparatively little annual change, as the principal areas requiring such treatment have long since been drained.

THE NEED FOR MORE FOOD

Man faces fully as serious a problem in the control of population growth as he does in the control of the atom. The explosive results of the one could as decisively destroy civilization as would the other. Hand in hand with the control of an exploding population must go the expansion of food-producing capabilities of the earth's people through the use of all the tools and techniques of an enlightened civilization.

The greatest shortages of food occur in the Far East and China. Each will, under the circumstances assumed, fail by approximately 50% in its ability to feed its people by the year 2000.

The deficiency can be reduced only by the most intensive effort in the improvement of agriculture, but, more than that, by the development of other natural resources through the labor of a country's workers, the products of which can be traded to nations with food surpluses.

All of the foregoing estimates for the future, inaccurate though they may prove to be, still serve to emphasize the seriousness of the problem and the need for an intensive program of expanding food production by all means. This includes irrigation and drainage in the humid areas, which constitute a large part of the earth's surface, particularly in the Far East, where the nations' capabilities for feeding their people are so seriously limited.

THE FUTURE OF IRRIGATION AND DRAINAGE PRACTICES IN HUMID AREAS

The need for increased food production throughout the world will compel the development of all arable areas. The cost and difficulty of obtaining irrigation water for the remaining nonproducing arid lands have increased greatly and new developments are definitely limited, including the potentiality of desalting sea water. Humid areas, on the other hand, offer far more opportunities for yielding greater food supplies through the efficient maintenance of soil moisture in the root zone. In the midcontinent United States, for example, large areas can be brought under irrigation.

With the expansion of irrigation in humid areas in the United States will come the need for additional drainage, but not in the same proportion. Present drainage systems may, with minor modifications, be made to serve for the removal of excess irrigation water as well as storm water.

Among the other regions of the world there are vast areas within the humid climatic belts which can be brought into food production or can be more efficiently utilized. A serious problem of mineral salts deposition on irrigated lands in West Pakistan resulting from inadequate drainage has been discussed elsewhere.[5] It is estimated that 100,000 acres per yr are being taken out of production and that 1,600,000 acres need restorative treatment. Philip P. Dickinson, F. ASCE, has stated in regard to irrigation in Ceylon that because of the maldistribution of rainfall and the abnormally high demand for water by rice, the island's principal crop, irrigation is essential to the future of that nation, which experiences an annual rainfall of well in excess of 100 in. over most of its area.

THE ROLE OF THE CIVIL ENGINEER

The geoagronomist and the civil engineer share a responsibility for expansion of the world's food production. The former determines what lands may be made available and gives advice and guidance in the farming of both new lands and of lands now under cultivation. This guidance will include instruction in land preparation, farming methods, planting, and the use of fertilizers, insecticides, and fungicides.

By providing proper irrigation and drainage the civil engineer guarantees that water in the right amount will always be available to the plant at its root zone. It is only by this combination of scientific effort of agronomist and engineer that there can be any hope of feeding the expanding world population.

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[5] Engineering News-Record, McGraw-Hill Publishing Co., Inc., New York, Vol. 163, No. 26, Dec. 24, 1959, p. 44.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3144

DUST PROPERTIES AND DUST COLLECTIONS

By B. Gutterman,[1] M. ASCE, and W. E. Ranz[2]

SYNOPSIS

Analytical methods have been developed that permit computation of reentrainment and backmixing characteristics of dust suspensions and dust layers. The methods are based on the physical properties of the dust and the fluid conveying or acting on the dust.

INTRODUCTION

This research was a basic study of some of the physical properties of dust suspensions and dust layers and the relation of these properties to dust collection. If the behavior of dust suspensions and dust layers in a collector were known in some detail, and if simple test procedures were available for predicting this behavior, costly trial and error designs, long developments, and faulty applications of dust collectors might be avoided. This investigation was made to establish the relationship between the physical properties of dust and critical pickup or entrainment velocity. In addition, an attempt was made to elucidate the mechanism or mechanisms responsible for backmixing of dusts, that is, to establish the relation between physical properties of dust and the distribution of dust concentration in a turbulent gas flow along a collecting surface.

Physical properties such as internal friction, surface friction, particle density and particle size, and cohesion were analyzed. Particle shape was specifically omitted because of its extreme variability. The shape factor, however,

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Note.—Published essentially as printed here, in July, 1959, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2088. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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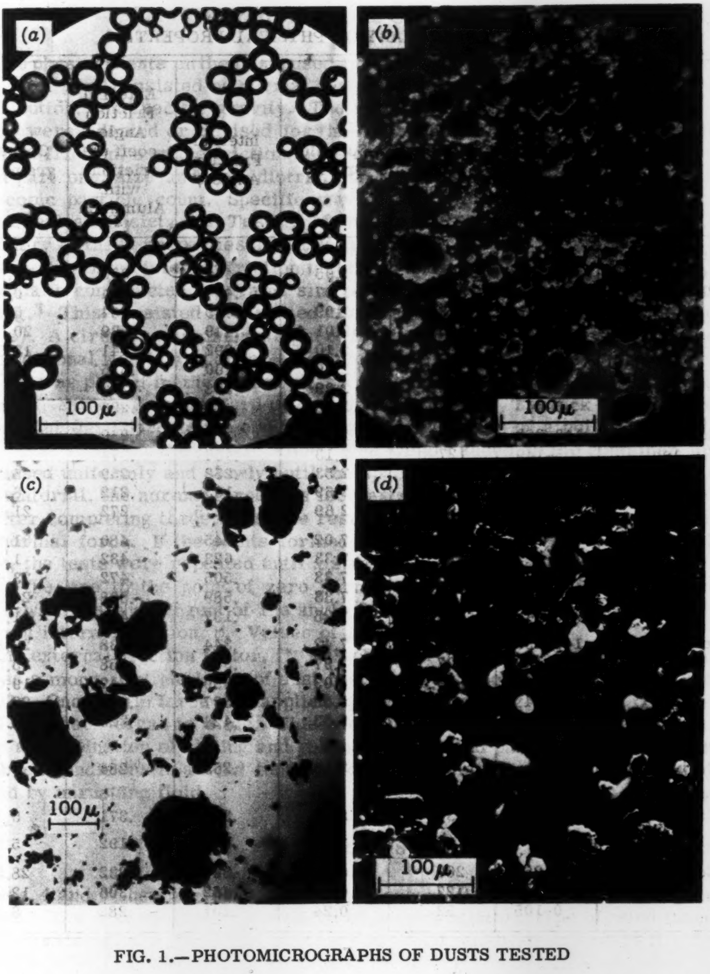
(a)
100μ
(b)
100μ
(c)
100μ
(d)
100μ

FIG. 1.—PHOTOMICROGRAPHS OF DUSTS TESTED

is not overlooked, but incorporated into such parameters as surface and internal friction.

Materials tested were sand, glass (Fig. 1a), tin, cork (Fig. 1d), carborundum, and fly ash (Fig. 1b). Photomicrographs of some of these dusts appear in Fig. 1. ((Fig. 1c) is of iron dust.) Order of magnitude values rather than precise quantities were obtained in order to get the broadest, most significant information from so general a study.

TABLE 1.—SUMMARY OF PHYSICAL PROPERTIES

Material	Size Range (Microns)	Average Particle Size (Microns)	Specific Gravity	Internal Friction Angle ψ	External Friction Angle coeff of friction with Aluminum f*	Cohesion grams/in^2
Union Carbide Fly Ash	420-590	505	1.95	.468	.300	13.5
	295-420	358	1.97	.467	.404	0.5
	250-295	372	1.99	.369	.354	5.4
	149-250	200	2.01	.449	.539	20.5
	105-149	127	2.10	.292	.441	13.5
	74-105	89.5	2.19	.400	.515	0.5
	44-74	59	2.29	.338	.297	14.5
	0-44	22	2.36		.256	14.5
Cottrell Fly Ash	149-250	200	2.18	.326	.230	8.5
	105-149	127	2.13	.436	.374	
	74-105	89.5	2.53	.252	.259	5.0
	44-74	59	2.69	.163	.212	6.0
	0-44	22	2.69	.0275	.272	21.0
Tin	149-250	200	7.02	.845	.480	31.0
	105-149	127	7.33	.623	.432	1.0
	74-105	89.5	7.33	.505	.472	33.0
	44-74	59	7.33	.589	.398	24.0
	0-44	22	7.25	.137	.405	31.0
Iron	149-250	200	7.42	.344	.338	29.5
	105-149	127	7.97	.347	.258	25.5
	74-105	89.5	8.03	.399	.342	19.0
	44-74	59	7.59	.367	.334	22.0
	0-44	22	7.33	.424	.326	10.5
#800 Carborundum	------	17	3.165	.263	.284	9.0
#400 Carborundum	------	37	3.17	.330	.371	5.0
Glass Beads	------	25.4	2.50	.246	.192	5.5
Cork	149-250	200	0.24	.307	.232	28.5
	105-149	127	0.24	.452	.306	12.0
	0-105	52	0.24	.401	.282	8.3

Notation.—The letter symbols adopted for use in this paper are defined where they first appear and are arranged alphabetically, for convenience of reference, in Appendix II.

PHYSICAL TESTS OF DUSTS

The physical tests on the dusts used in this study were divided into two parts. The first part consisted of normal tests such as particle size, particle size distribution, and specific gravity. The second part consisted of special tests which were adapted or devised for this research. These special tests were internal friction, surface friction, and cohesion.

In part one, size and size distribution were measured by sieving and microscopic particle count. Specific gravity tests were conducted according to the American Society for Testing Materials (ASTM), Designation D-854. A summary of these properties appears in Table 1.

In part two, the internal friction factor, ψ, and cohesion were measured with a specially constructed apparatus similar to the direct shear test used in soil testing.[3] This consisted of a milled aluminum block with a plastic slide (see Fig. 2). A circular aluminum weight of fifty grams was used as the surcharge and additional weight was added to the normal force when needed. The particle sample was placed in the shear box by means of a funnel and sharply rapped three times to obtain a standard degree of densification. The lock was released and weight was added to the container, which, by means of a pulley, exerted a horizontal force on the plastic slide. The weight added to the container was increased uniformly and slowly until the specimen failed. With the addition of new material, the normal force was increased and the test repeated.

After completing three tests, the results were plotted as shear stress versus normal force. If the points formed a straight line the test was accepted. If not, the tests were repeated until a straight line was obtained. By extrapolating the line to the point of zero normal force, the value of cohesion was found, whereas the tangent of the angle formed by the straight line gave the angle of internal friction, ψ, Values of ψ are found in Table 1.

The external friction factor, f^*, was obtained by pulling an aluminum block across a mono-particle layer of dust resting on a smooth aluminum plate (see Fig. 3). Once again force was applied by means of adding water to a container until the block began to slide. This method of determining a friction factor gives a combination of sliding and rolling friction and particularly resembles observed conditions in a wind tunnel whose particulate matter is picked up or moved by onrushing fluid.

APPARATUS

Figs. 4 and 5 show the test apparatus and schematic diagram of the sampling system. The test equipment consisted of a recirculatory type of wind tunnel with a horizontal aluminum test section that was 2-in.-by-5-in.-by-16 ft. The air fan was belt driven and had an adjustable butterfly valve mounted directly above the discharge. Mounted on top of the channel, at 5 ft intervals, were plastic windows that served as observation and sampling ports. Fully developed

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[3] "Soil Testing for Engineers," by W. T. Lambe, John Wiley and Sons, Inc., New York, 1951, p. 88.

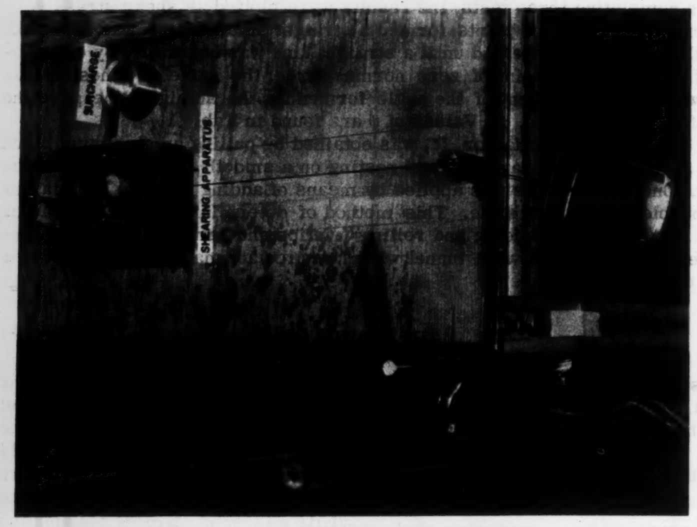
SURCHARGE
SHEARING APPARATUS

FIG. 2.—APPARATUS FOR MEASURING INTERNAL FRICTION FACTOR AND COHESION

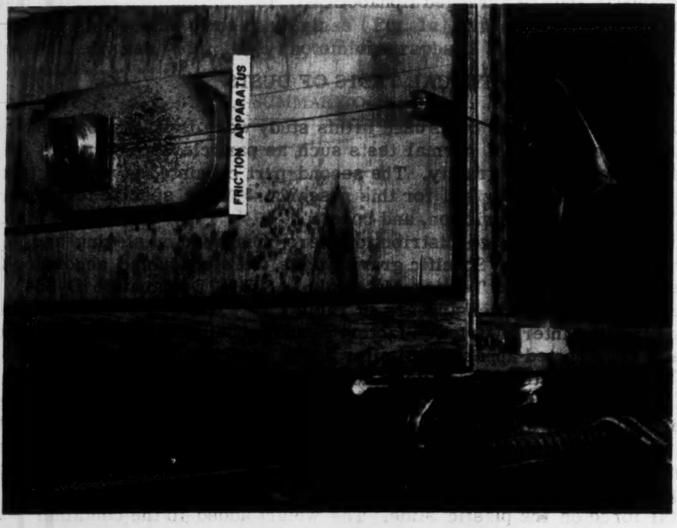
FRICTION APPARATUS

FIG. 3.—APPARATUS FOR MEASURING FRICTION FACTOR

turbulence was induced by means of saw tooth spoilers placed in the entrance section of the tunnel.

The sampling system comprised a wet test meter, rotameter, vacuum pump, and appropriate pressure sensing devices ($\frac{1}{2}$-in. and 1-in. inclined water manometers that were used in conjunction with miniature impact tubes for measuring velocities in the system. This same system was also used for measuring

FIG. 4.—TEST APPARATUS

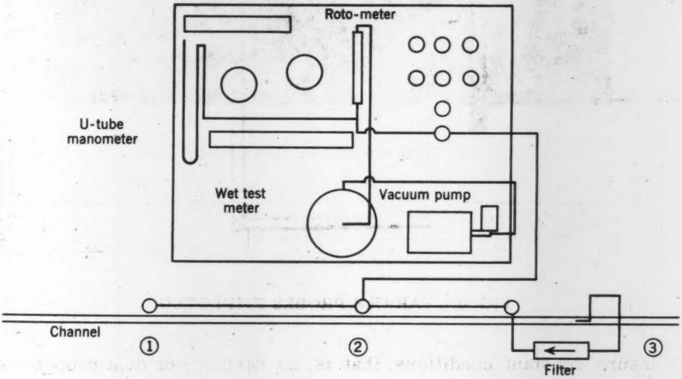

FIG. 5.—SAMPLING SYSTEM

concentration gradients when the sampling filter was connected to the vacuum line. Various probes were used according to their specific need (Fig. 6). Probe number one was used for measuring velocity of dust laden streams, number two for sampling, and number three was used for velocity measurements.

The sampling probes described had a diameter of approximately 100 times the size of the largest particle tested. These probes were connected under vacuum to the sample holder (Figs. 7 and 8). The sample holder, which is different from the thimble type in current use, was made to use glass fiber filter paper. The glass fiber filter paper was chosen because it absorbed no moisture.

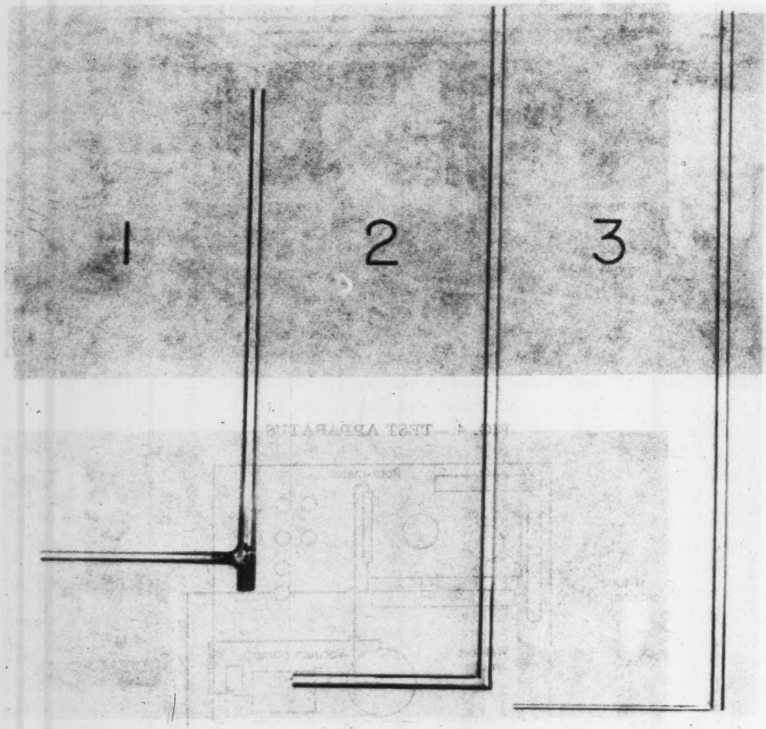

FIG. 6.—VARIOUS PROBES EMPLOYED

To insure constant conditions, that is, no variance of dust properties and gas stream properties due to temperature and humidity, an air conditioner and dehumidifier were installed in the laboratory. By means of the thermostatic control devices on the conditioner, the ambient temperature was kept at 75° F

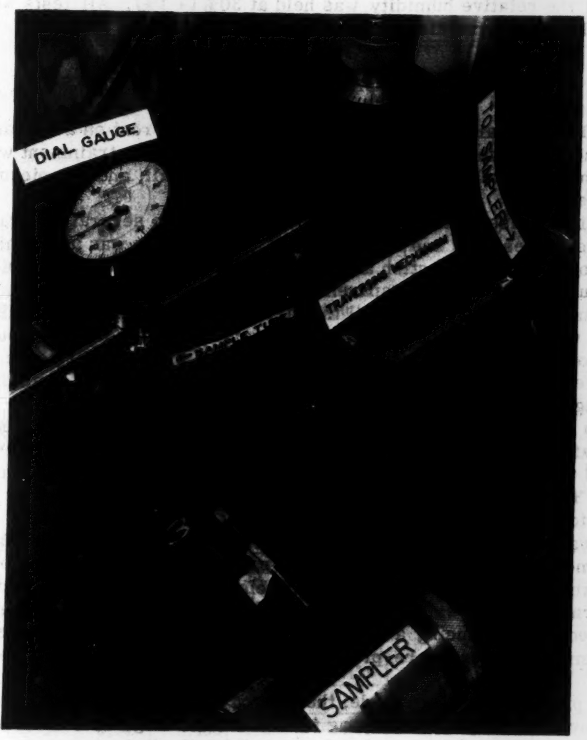
DIAL GAUGE
TO SAMPLER
TRAVERSING MECHANISM
SAMPLER

FIG. 7.—SAMPLE HOLDER ARRANGEMENT

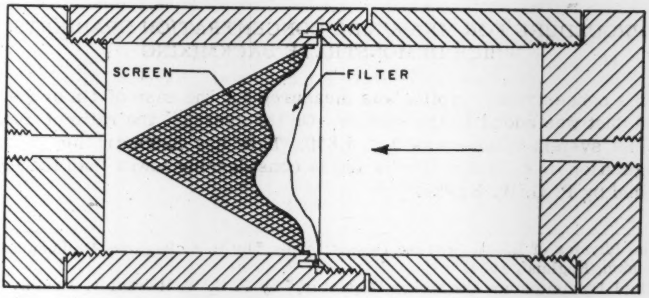
SCREEN
FILTER

FIG. 8.—SAMPLE HOLDER

(± 3°) and the relative humidity was held at 30% (± 1%). All tests were conducted under these conditions.

PROCEDURE FOR REENTRAINMENT STUDIES

The velocity of air required to move a particle at rest on a surface is defined as the pickup velocity. Experiments concerning reentrainment were carried out in two parts; (1) pickup from the bottom wall, and (2) pickup from a particle bed.

In the first phase, graded dusts were separated into several size ranges and each range represented one test specimen. A specimen was uniformly spread on the floor of the test section and observed by means of a telescopic microscope mounted above the transparent channel top. The air velocity in the section was increased from zero until the particles began to move. Velocity measurements were recorded when particle movement occurred throughout the entire section. Data on pickup from the bottom wall are summarized in Table 2. It is interesting to note that initial movement was marked by a slight rolling and sliding action.

Pickup from the particle bed was accomplished by inserting a grooved aluminum plate and attaching it to the floor of the test section (Fig. 9). The groove was filled with particles and the top of the particles was leveled with the plate surface. This time, the particles had to roll or slide over the other particles in order to move and wholesale movement occurred only when an upstream particle was lifted into the stream and then impinged upon the bed surface. This bombardment caused one, two, or more particles to fly out of the bed and to restrike the surface releasing more particles. This action was noted by a series of craters that seemed to be blasted in the particle surface. Bombardment was proved by introducing a dissimilar material upstream and observing the bed. When the craters were produced, a foreign particle was lodged in each hole.

When pickup tests were made, the return line of the recirculation system was removed and replaced with bag filters. This change was necessary in order to keep recirculated particles from striking the bed.

The observations previously noted are identical to those of R. A. Bagnold[4] and confirm his conclusion about conditions necessary for particle movement.

PROCEDURE FOR MEASURING CONCENTRATION GRADIENTS WHICH DEMONSTRATE BACKMIXING

When a concentration profile was measured for the case of tin or sand, the selected dust was added to the system. On the average, the ratio of dust volume to the system volume was 1 in 4,850. This low concentration prohibited any coagulation or collision effects and is considerably lower than the ½% recommended by P. G. W. Hawksley.[5]

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[4] "The Physics of Blown Sand and Desert Dunes," by R. A. Bagnold, Methuen and Co., Ltd., London, England, 1954.

[5] "Fluid Dynamics and the Stokes Diameter," by P. G. W. Hawksley, Monthly Bulletin, British Coal Utilization Research Assn., Vol. XV, No. 4, Review Series 102, April, 1951, p. 129.

TABLE 2

Ave d_p (Microns)	d_p (ft) x 10^4	Sg	ρ_p	f*	$d_p{}^3 \rho_p f^*$ x 10^{12}	$C_D \mathbf{R}^2$	$\mathbf{R}$	τ_o (comp) x 10^4	τ_o (obs) x 10^4
U. C. Fly Ash									
22	0.72	2.36	4.55	0.256	0.435	0.28	0.0117	2.98	14.0
59	1.935	2.29	4.21	0.297	9.45	6.1	0.252	8.96	8.3
89.5	2.93	2.19	4.22	0.515	54.6	35.3	1.28	19.75	4.57
127	4.16	2.1	4.05	0.441	128.5	83	2.8	20.65	4.57
200	6.56	2.01	3.87	0.539	587.	379	9.5	30.4	4.29
272	8.93	1.99	3.94	0.354	991.	640	14.4	23.9	4.29
357.5	11.7	1.97	3.80	0.404	2710.	1750.	29.6	29.5	5.15
480	15.7	1.95	3.76	0.300	4330	2800	40.8	22.1	4.28
Total Mix				0.494					
Cottrell Fly Ash									
22	0.721	2.69	5.22	0.272	0.530	0.35	0.0147	3.79	8.29
59	1.935	2.69	5.22	0.212	7.98	5.2	0.217	7.68	7.72
89.5	2.93	2.53	4.8	0.259	31.3	20.1	0.76	11.88	6.29
127	4.16	2.13	4.125	0.374	111	71.7	2.16	16.55	4.87
200	6.56	2.18	4.225	0.230	274	192	5.5	17.65	3.14
Total Mix				0.425					
Carborundum									
37	1.21	3.17	6.15	0.371	4.25	2.74	0.113	10.2	54.2
18	0.59	3.17	6.15	0.284	36	0.233	0.0098	3.74	36.4
Glass Beads									
28	0.918	2.5	4.84	0.192	7.18	0.463	0.02	3.14	10
Cork									
100	3.28	0.24	0.466	0.282	4.63	2.98	0.124	1.44	2.28
127	4.16	0.24	0.466	0.306	10.3	6.65	0.28	2.15	2.28
200	6.56	0.24	0.466	0.232	30.5	19.7	0.75	2.4	2.28
Iron Dust									
22	0.721	7.33	14.2	0.326	1.12	0.724	0.0305	7.8	20.9
59	1.935	7.59	14.7	0.334	35.45	22.9	.81	28.7	20.2
89.5	2.93	8.03	15.55	0.342	134	86.5	2.80	43.2	16.9
127	4.16	7.97	15.4	0.258	286	185	5.8	44.3	17.2
200	6.56	7.42	19.35	0.338	1370	884	18	55.5	16.3
Total Mix				0.395					
Tin Dust									
22	0.721	7.25	13.9	0.405	2.105	1.355	0.0565	14.45	12.87
59	1.935	7.33	14.1	0.398	40.5	26.1	0.965	35.4	11.7
89.5	2.93	7.33	14.1	0.472	167	108	3.9	60.8	20.3
127	4.16	7.33	14.1	0.432	437	282	7.5	57.6	16.6
200	6.56	7.02	13.5	0.480	1825	1180	22.2	71.0	25.2
Total Mix				0.349					

After the dust was thoroughly mixed in the system, the sampling lines were flushed by a sharp air blast and immediately connected to the sampler whose flow rate had been set previously. After the alloted sampling time the collected sample was removed and weighed and an equal weight of material was added to the system. Each sampling position was alternately tested with a selected datum level and deficiencies in the total concentration were corrected by increasing the amount of material added after each removal.

When concentration gradients were made for the glass beads, the mixing chamber was removed and the system was operated under vacuum with the discharge being routed to the bag filters. A continuous feed of the beads was introduced upstream by means of a burette and hose clamp system. Concentrations at various levels were made in an identical manner to those of tin and sand. Data on a concentration gradient for glass beads (Fig. 10a), tin, and sand are shown in Fig. 10.

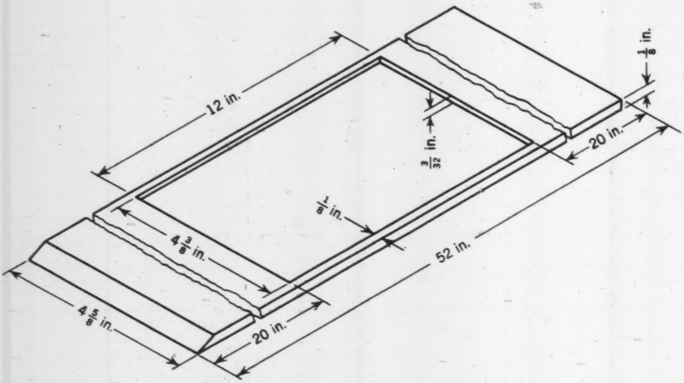

FIG. 9.—CHANNEL INSERT FOR PARTICLE BED

The analysis that follows is divided into two parts: The first deals with reentrainment, and the second with backmixing by diffusion and bounce.

REENTRAINMENT

Wall Pickup Theory.—The fact that particles rolled or slid in moving on a flat plate led to the observation that friction played a part in resisting the driving force of the air stream. The laminar sublayer on the wall of a channel which carries a turbulent gas flow, has a thickness of an order given by O. W. Eshbach:[6a]

$$\delta' \frac{\sqrt{\tau_0/\rho_g}}{\nu_g} \approx 11.4 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

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[6] "Handbook of Engineering Fundamentals," by O. W. Eshbach, 2nd ed., John Wiley and Sons, Inc., New York, 1952 (a) pp. 6-45 (b) pp. 6-10.

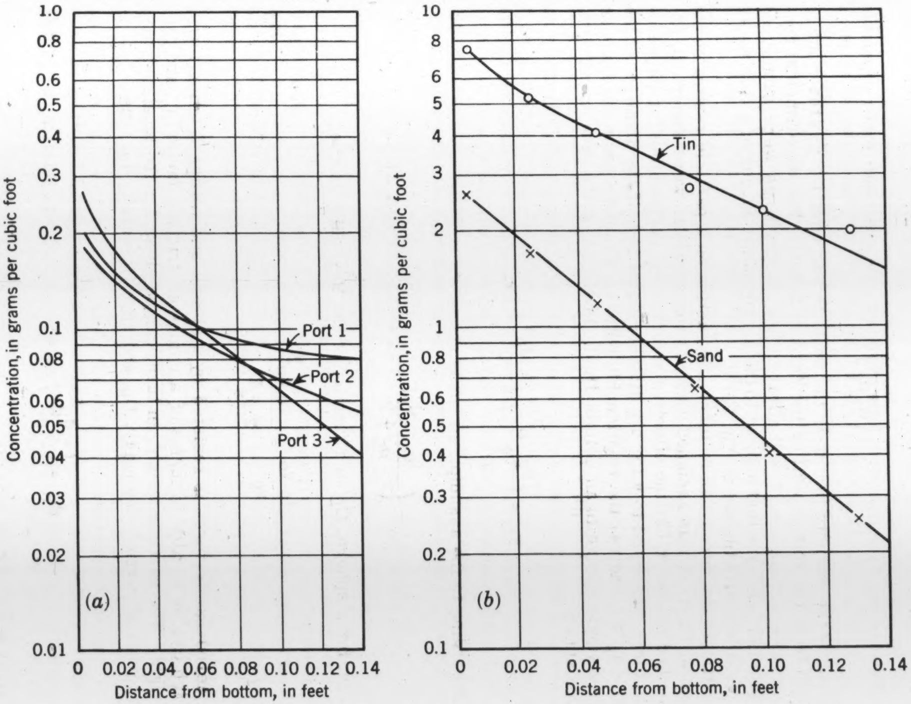

FIG. 10.—CONCENTRATION PROFILES

in which δ' is the boundary layer thickness in feet, τ_o is the shear stress at the wall in pounds per square foot, ρ_g is the gas density in pound-second squared per foot to the fourth power, and ν_g refers to the kinematic viscosity of the gas in square feet per second. As an approximation, the velocity gradient in this layer is equal to the velocity gradient at the wall. Thus,

$$v = \left(\frac{\tau_o}{\nu_g\,\rho_g}\right) y \quad \ldots\ldots \quad (2)$$

approximates the velocity field near the wall. In this equation y is the distance measured from the bottom wall of the channel in feet. For experimental conditions δ' was a minimum of 250 microns and all particles were submerged in this laminar sublayer and subject to the aerodynamic forces of a flow whose velocities are approximated by Eq. 2.

To estimate the aerodynamic force, F_D, acting on a particle resting on the test section floor, it was assumed that (1) the particles were spherical, (2) the floor had no effect, (3) the effective velocity, v_o, responsible for the aerodynamic force was equal to v for a y distance equal to half the diameter of the particle, $(d_p/2)$, and (4) the relationship between F_D and v_o was the same as that for a single sphere in an infinite fluid. In terms of the drag coefficient, C_D, for a sphere

$$F_D = \left(\frac{\rho_g\,{v_o}^2}{2}\right)\left(\frac{\pi\,{d_p}^2}{4}\right) C_D \quad \ldots\ldots \quad (3)$$

where according to the assumptions made above

$$v_o = \left(\frac{\tau_o}{\nu_g\,\rho_g}\right)\frac{d_p}{2} \quad \ldots\ldots \quad (4)$$

and where, for a sphere, C_D is a function of the Reynold's number,

$$R = \frac{d_p\,v_o}{\nu_g} = \frac{\tau_o\,{d_p}^2}{2\,{\nu_g}^2\,\rho_g} \quad \ldots\ldots \quad (5)$$

The aerodynamic force is resisted by a sliding and rolling friction force, F_R, which can be written in terms of a combined friction factor, f*, and particle weight. The particle weight, in turn, can be given for the idealized particle in terms of particle diameter, D_p, and specific weight, γ_p, thus,

$$F_R = \left(\frac{\pi\,{d_p}^3}{6}\right)\gamma_p\,f^* \quad \ldots\ldots \quad (6)$$

The critical value of flow velocity, as measured by τ_o occurs when

$$F_D = F_R \quad \ldots\ldots \quad (7)$$

Combining Eqs. 3 through 7, it is found that

$$C_D \mathbf{R}^2 = \frac{4}{3}\left(\frac{d_p{}^3\ \gamma_p\ f^*}{\nu_g{}^2\ \rho_g}\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

a quantity which depends only on the physical properties of the air and dust in question, in particular, on the special friction factor f*.

The relationship between **R** and C_D **R**^{2} for spheres is well known.[7] To estimate the critical value of τ_o from known values of f*, it is now only necessary to compute the value of C_D **R**^{2}, refer to **R** - C_D **R**^{2} plot, (see Fig. 11), obtain the value of **R** and compute τ_o through the use of Eq. 10. As many quantities in the equations are constant for a given condition, the equation reduces to the form of

$$C_D \mathbf{R}^2 = 0.646 \times 10^{12} \left(d_p{}^3\ \rho_p\ f^*\right) \quad \ldots\ldots\ldots\ldots \quad (9)$$

$$\tau_o = 1.325 \times 10^{-10}\ \mathbf{R}/d_p{}^2 \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

A comparison of shear stresses associated with wall pickup and critical shear stresses computed from friction data is shown in Table 2. Although the results do not coincide exactly with theory, it is felt that a proper order of magnitude has been established. The many difficulties encountered in operation may have contributed to the source of error. Notable are the inability to preserve a mono-particle layer, the use of graded aggregate causing only larger particles to be contacted by the normal force during friction tests, and improper manometer adjustment in establishing τ_o.

An experimental study on transport velocity has been made by J. Baliff, L. Greenburg and A. C. Stern.[8] Conveniently, these men record scour or clean out velocities of some industrial dusts. Because none of their test dusts match any of those examined in this investigation, values of friction factor were assumed, based on experience with similar types of materials used on this project. With the assumption of the friction factor and being given the particle size and density of the material, the theoretical shear stress was computed. From a relation of shear stress and velocity for the test equipment, the average velocity was computed and compared to those observed by Stern. Comparisons are shown in Table 3. Good agreement occurs, as is noted in Table 3. A large error cannot be incurred as friction factors of the materials tested ranged from two-tenths to five-tenths.

Dust Layer Pickup Theory.—In a bed of particles, a particle on top, acted upon by aerodynamic forces, takes the path of least resistance and begins to slide or roll between adjacent downstream particles. The forces opposing this motion are friction between the particles (grain interlock), cohesion (if any exists), and gravity. When the particle approaches a higher elevation, the driving force has increased considerably as velocity increases with height and interlock resistance is reduced to sliding or rolling friction of one particle on

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[7] "Fluid and Particle Mechanics," by C. E. Lapple, et al., Univ. of Delaware, Newark, Del., p. 290.

[8] "Transport Velocities for Industrial Dusts, An Experimental Study," by J. Baliff, L. Greenberg, and A. C. Stern, Monthly Review, Div. of Hygiene and Safety Standards, New York State Dept. of Labor, Vol. 28, No. 12, December, 1949.

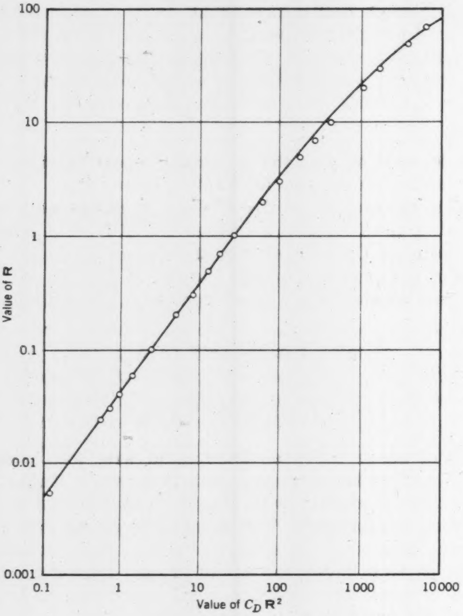

FIG. 11

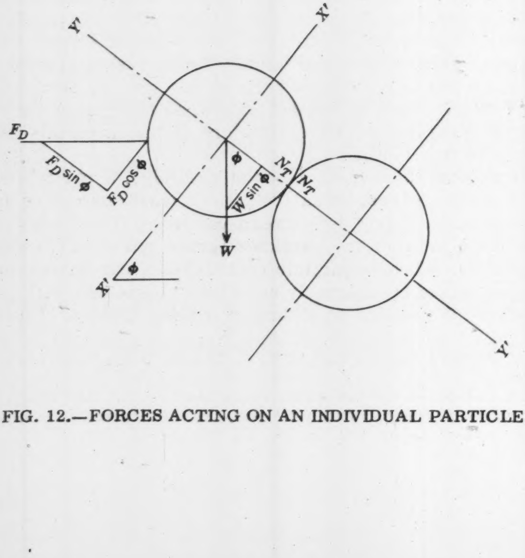
Y'
X'
F_D
$F_D \sin\phi$
$F_D \cos\phi$
ϕ
$W \sin\phi$
N_T
N_T
W
ϕ
X'
Y'

FIG. 12.—FORCES ACTING ON AN INDIVIDUAL PARTICLE

another. Movement occurs rapidly because of increased forces until the particle hits on an adjacent particle and, like leaving a springboard, jumps out into the stream. Once a few particles are dislodged, a chain reaction of bombardment occurs, and the particle bed disintegrates.

Theoretically, the particles are assumed to be spherical, and the friction factor used is that of internal friction or grain interlock, ψ. The effective velocity responsible for the aerodynamic force is again assumed to be $v_o = \left(\frac{\tau_o}{\nu_g \rho_g}\right)y$.

TABLE 3

Description Method of Production	Sample 1 Foundry non-ferrous wheel abrator	Sample 2 Metal (alum & Bronze Grinding	Sample 3 Steel Shot Blast Cabinet
d_p (microns)	120	100	125
d_p (feet)	4.16×10^{-4}	3.28×10^{-4}	4.16×10^{-4}
d^2_p (sq ft)	17.3×10^{-8}	10.38×10^{-8}	17.3×10^{-4}
d^3_p (cu ft)	71.9×10^{-12}	35.2×10^{-12}	71.9×10^{-12}
Sg	3.02	6.34	6.85
$\rho \left(\frac{\text{lb sec}^2}{\text{ft}^4}\right)$	5.85	12.3	13.3
f*	0.4	0.4	0.3
$d^3_p \rho_p f^*$	168.2×10^{-12}	173×10^{-12}	287×10^{-12}
$C_D \mathbf{R}^2$	103.7	111.5	185
$\mathbf{R}$	3.43	3.5	5.4
N_{Re}/d^2_p	10.85×10^6	34.7×10^6	31.2×10^6
τ_o computed lb per sq ft	26.3×10^{-4}	461×10^{-4}	41.4×10^{-4}
Velocity computed (ft per min)	1360	1800	1720
Velocity observed by Stern et. al. (ft per min)	1650	1560	1900

A pictorial representation of forces acting on an individual particle appears in Fig. 12. In order for a particle to move over another, the ball must roll or slide up the adjacent particle. This is equivalent to moving up an inclined plane. In addition to the necessity of rolling up and over the downstream particle, friction between the particles is encountered. This friction can be thought of as an additional inclination over which the particle must move. The total inclination must therefore be the sum of the constant contact angle plus the additional inclination due to friction. The angle ϕ, as noted in Fig. 12, represents this total inclination.

TABLE 4.—PICKUP FROM

Dust	d Microns	$d(ft) \times 10^4$	Sg	ρ_p	$\tan \phi\ (\psi)$
U.C.F.A.	505	16.58	1.95	3.78	0.468
	358	11.72	1.97	3.82	0.467
	272	8.92	1.99	3.85	0.369
	200	6.56	2.01	3.95	0.449
	127	4.17	2.10	4.07	0.292
	89.5	2.94	2.19	4.25	0.400
	59.	1.94	2.29	4.43	0.338
	22.	0.72	2.36	4.57	----
Cottrell F.A.	200	6.56	2.18	4.22	0.326
	127	4.17	2.13	4.13	0.436
	89.5	2.95	2.53	4.80	0.252
	59	1.94	2.69	5.21	0.163
	22	0.72	2.69	5.21	0.0275
Tin	200	6.56	7.02	13.6	0.845
	127	4.17	7.33	14.2	0.623
	89.5	2.95	7.33	14.2	0.505
	59	1.94	7.33	14.2	0.589
	22	0.72	7.25	14.05	0.137
Iron	200	6.56	7.42	14.35	0.344
	127	4.17	7.97	15.45	0.347
	89.5	2.95	8.03	15.55	0.399
	59	1.94	7.39	14.7	0.367
	22	0.72	7.33	14.2	0.424
Cork	200	6.56	0.24	0.466	0.307
	127	4.17	0.24	0.466	0.452
Carborundum	37	1.21	3.17	6.15	0.330
	18	0.59	3.16	6.15	0.263
Glass	28	0.918	2.50	4.85	0.246

$$F'_x = 0$$

$$F'_x = F_D \cos \phi - W \sin \phi \quad \ldots\ldots\ldots\ldots (11a)$$

or

$$F_D = W \tan (\phi) \quad \ldots\ldots\ldots\ldots\ldots\ldots (11b)$$

Tan ϕ is identical to the friction factor obtained from the direct shear test and is denoted as ψ. In an idealized bed of rhombohedrally packed spheres, which show no cohesion, $\psi = \tan \phi = \tan 30°$. A theoretical determination of the critical τ_o is as before (Eqs. 9 and 10), where now f^* has the physical meaning of ψ or $(\tan \phi)$.

Theoretical Results.—A summary of theoretical shear stresses based on friction factor measurements and observed shear stresses for pickup from particle beds is shown in Table 4. The computed values of shear stress are generally higher than those observed and may be as much as three times larger. Although this large discrepancy occurs, it is felt that the values fall within

PARTICLE LAYER

ϕ degrees	$d_p^3 \rho_p \times 10^9$	$C_D R^2$	R	$R/d^2 \times 10^{-8}$	τ_o calc. $\times 10^4$	τ_o obs. $\times 10^4$
25.09	8.04	5200	62	0.226	29.95	13.0
25.05	2.89	1870	31.2	0.226	29.95	12.0
20.50	1.01	654	14.3	0.180	23.8	12.2
24.20	0.50	323	8.3	0.193	25.6	10.7
16.28	0.0857	55.3	1.90	0.110	14.8	12.0
21.80	0.0435	28.1	1.04	0.120	15.9	10.0
18.68	0.0109	7.08	0.295	0.0785	10.4	13.9
----	----	----	----	----	----	----
18.05	0.388	251	6.9	0.16	21.2	7.4
23.55	0.1295	83.6	2.73	0.1575	20.9	11.4
14.14	0.0308	19.9	0.76	0.088	11.7	19.5
9.26	0.0062	4.02	0.168	0.0447	5.92	----
1.575	0.00002	0.0129	0.0055	0.0106	----	----
40.2	3.1	2010.0	32.9	0.765	101.5	59.8
31.85	6.39	413.0	10.2	0.590	78.4	48.8
26.80	0.183	118.0	3.72	0.431	27.1	45.5
30.45	0.061	39.5	1.4	0.373	49.5	27.3
7.79	0.0027	1.75	.074	0.145	19.0	28.0
19.00	1.39	900	18.2	0.424	55.2	37.5
19.22	0.386	250	6.9	0.392	52.0	32.5
21.75	0.158	102	3.21	0.372	49.3	23.2
20.15	0.0393	25.4	0.94	0.250	33.2	23.5
22.98	0.000843	0.86	0.036	0.0694	18.7	16.25
17.08	0.0403	26.1	0.97	.0226	3.0	6.4
24.30	0.01522	9.87	0.41	.0237	3.14	10.5
18.25	0.00359	2.32	0.097	6.65	8.8	19.2
14.72	0.000332	0.215	0.0091	2.62	3.5	----
13.81	0.000922	0.597	0.0252	3.0	4.0	21.2

a reasonable order of magnitude. If velocity is used as a criterion of comparison instead of shear stress, the computed and observed values are closer together. A complete velocity relationship has been purposely omitted in order to preserve a general nature of solution to this type of problem. A comparison of some observed and computed values of velocity and shear stress appears in Table 5.

In some cases, particularly in the smallest size range of fly ash and carborundum dust, experimental values of shear stress could not be taken because velocities required for pickup exceeded that of the equipment. The velocity of air required to keep the material airborne, however, is considerably less than that required to pick it up from the bed.

Difficulties encountered are similar to those of wall pickup with a few exceptions. The notable exception is the inability to clean the system before the start of the tests. If one, two, or more individual particles tore loose from the upstream housing, these particles could have caused an erroneous reading. Extreme care was taken to avoid such incidents, but the possibility still existed. Once again it was difficult to observe the very small particles.

Pickup from particle layers has also been studied by A. Shields who treats sediment entrained from river bottoms. The Shields[9] entrainment function, as shown in Fig. 13, is usually presented with abscissa values from one to 1000. The values obtained in this study do not cover the entire range of Shields' work but start in the laminar range of values less than one and overlap slightly.

Essentially, the theory for this work and that of Shields are derived in an identical manner. Extension of the theory has been made here by an attempt to define explicitly the Shields resistance and shape parameter in terms of an internal friction factor ψ.

Two sets of points are shown in Fig. 13. The squares represent the ordinate parameter as suggested by Shields and the circles represent a corrected Shields' parameter using the internal friction factor ψ. The closer agreement between the circles and Shields' curve would, according to the present theory, indicate that ψ for sediment transport is of the order of unity. Whereas both sets of points might be considered to be of the proper order of magnitude, it is felt that closer agreement with Shields might be obtained if the Shields parameter is corrected for internal friction factor. The value of ψ should be obtained in a manner consistent with its use. For example, in this study, ψ is obtained in an air atmosphere where small amounts of moisture will affect its value through a change in cohesion. If ψ is obtained under water, viscous effects may also alter its value.

BACKMIXING

Backmixing by Diffusion.—
Theory.—

$$\frac{\partial C}{\partial t} + u\,\frac{\partial C}{\partial x} + v\,\frac{\partial C}{\partial y} + V_t\,\frac{\partial C}{\partial y} = D_{eff}\left(\frac{\partial^2 C}{\partial x^2} + \frac{\partial^2 C}{\partial y^2}\right) \quad \ldots\ldots\ldots \quad (12)$$

is the differential equation for dust concentration in a dusty gas moving in a two-dimensional flow, where the particles are acted upon by a separating force that moves them in a negative y-direction at a terminal velocity, V_t, and where there is a backmixing effect that can be written in terms of an effective diffusion coefficient, D_{eff}, the coefficient D_{eff}, can be, and is, a function of x and y. In Eq. 12, C is the concentration in grams per cubic foot.

In the system under study, an x-distance will eventually be reached where $\frac{\partial}{\partial t}$ and $\frac{\partial}{\partial x}$ are zero. Because v is also zero, Eq. 12 reduces to

$$V_t\, C = D_{eff}\,\frac{\partial c}{\partial y} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

and the flux of particles toward the surface at any level is exactly balanced by backmixing. In Eq. 13, the terminal velocity can be computed. The concentration and concentration gradient can be obtained from a plot of concentration versus distance from the wall. An "experimental" diffusion coefficient can then be computed and compared to theoretical values computed on the basis of different backmixing mechanisms.

A logical mechanism for backmixing of small particles is turbulent diffusion. If backmixing occurs only by turbulent diffusion, that is, the particles follow the turbulent gas motion,

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[9] A. Shields, cited by H. Rouse in "Engineering Hydraulics," John Wiley and Sons, Inc., New York, 1949, p. 789.

$$D_{eff} = D' \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

in which D' is the turbulent diffusion coefficient of the gas.

TABLE 5

Material	Average Size (Microns)	Computed Critical Velocity, in ft per sec	Observed Critical Velocity, in ft per sec	$\frac{\text{Computed } v_o}{\text{Observed } v_o}$	$\frac{\text{v Computed = 1.0}}{\text{v Observed (idea)}}$
U.C.F.A.	505	22.2	13.0	2.3	1.71
	59	11.1	13.6	1.59	0.82
Cottrell F.A.	200	17.5	8.6	2.87	2.04
	89.5	12.0	17.4	0.60	0.69
Tin	200	42.5	32.5	1.70	1.31
	22	17.0	21.8	0.68	0.61
Cork	200	4.5	7.6	.47	0.590
	127	4.6	11.3	.314	0.411
Carborundum	37	10.0	17.0	.46	0.59
Glass	28	5.4	17.5	0.190	0.320

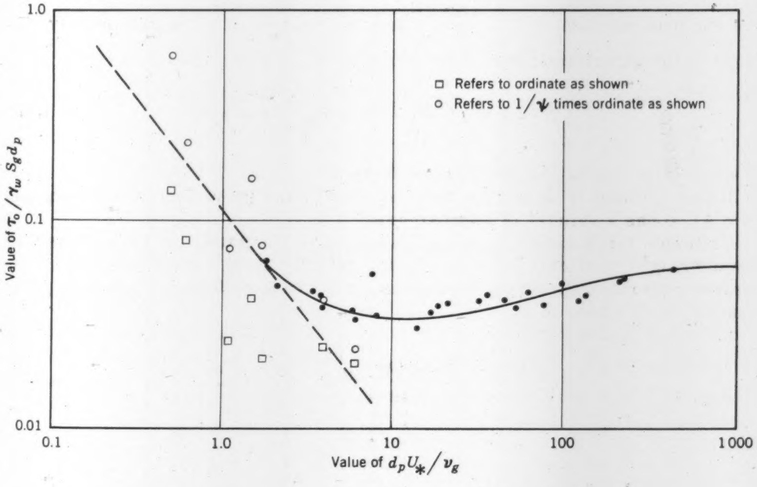

FIG. 13.—SHIELD'S ENTRAINMENT FUNCTION

Because D' is related to μ', the turbulent or eddy viscosity which can be computed from velocity field data, there are relatively simple methods of estimating D' in many cases of practical interest. D' is related[6b] to μ' by

$$D' = \mu' / \rho_g \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which μ' is defined by the shear stress,

$$\tau = \mu' \frac{du}{dy} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

Usually, the velocity field can be given in logarithmic form[10] over extended y-distances as

$$\frac{u}{u*} = \frac{1}{b} \ln \frac{y\, u*\, \rho_a}{\mu\, g} + \text{constant} \quad \ldots\ldots\ldots\ldots \quad (17)$$

in which $u* = \sqrt{\tau_o/\rho_g}$

and

$$\frac{du}{dy} = \frac{u*}{b\, y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

in which u is the velocity in the x-direction and u^* is the friction velocity. Combining Eqs. 15, 16, and 18

$$D' = \frac{\tau}{\rho_g} \frac{dy}{du} = \frac{\tau\, b\, y}{\rho_g\, u*} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

For the present case, $\tau = \dfrac{\tau_o\,(A - y)}{A}$ and D' in terms of b, the reciprocal of the slope of the logarithmic plot of the velocity is

$$D' = b\, y\, u* \frac{(A - y)}{A} = b\, y \sqrt{\frac{\tau_o}{\rho_g}}\, \frac{(A - y)}{A} \quad \ldots\ldots\ldots \quad (20)$$

in which A is one half of the channel depth.

It is noted that D' is zero at the wall (y = 0) and in the center of the channel (y = A), b has a value[11] of approximately 0.4.

Criterion for Backmixing by Diffusion.—J. P. Longwell and M. A. Weiss, by considering[12] particles in an oscillating velocity field, have developed a convenient criterion concerning conditions for particle diffusion. D_{eff} is approximately of the order of

$$\frac{D_{eff}}{D'} = \frac{\left(18\ \mu_g/\rho_g\, d_p^2\right)^2}{\omega^2 + \left(\dfrac{18\ \mu_g}{\rho_g\, d_p^2}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

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[10] "Engineering Applications of Fluid Mechanics," by J. C. Hunsaker and B. G. Rightmire, McGraw-Hill Book Co., Inc., New York, 1947, p. 143.

[11] "Essentials of Fluid Mechanics," by L. Prandtl, Hafner Publishing Co., New York, 1952, p. 121.

[12] "Mixing and Distribution of Liquids in High-Velocity Air Streams," by J. P. Longwell and M. A. Weiss, Industrial and Engineering Chemistry, Vol. 45, March, 1953, p. 667.

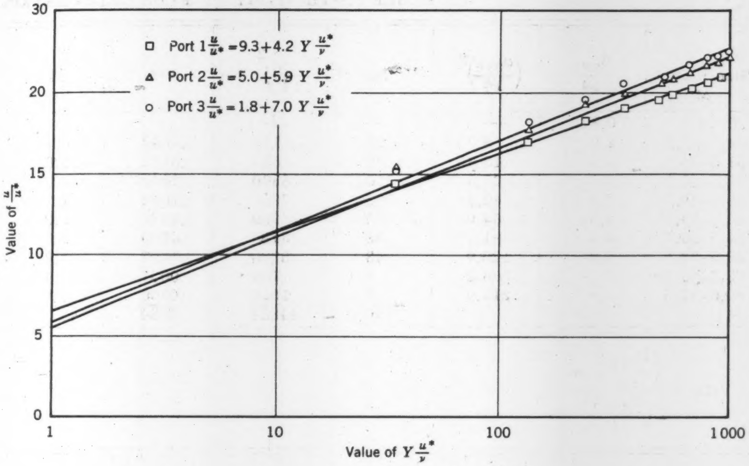

FIG. 14.—TYPICAL VELOCITY DISTRIBUTION

TABLE 6

Material	Size Range (Microns)	Sg	d_{ave} Microns	Diffusivity Ratio $\frac{D_{eff}}{D'}$
Sand	30-165	2.64	96	.0061
Spherical Tin	30-90	7.3	53.5	.0146
Glass Beads	3.25-39	2.50	25.4	.75

TABLE 7.—SAMPLE PORT NO. 3

Position	Material-Glass		Material-Sand		Material-Tin	
	Observed D sq ft per sec	Computed D sq ft per sec	Observed D sq ft per sec	Computed D sq ft per sec	Observed D' sq ft per sec	Computed $D_{eff.}$ sq ft per sec
1	.00154	----	.098	----	.0157	----
2	.0123	.0149	.089	.0166	.032	.0135
3	.0153	.0150	.084	.0157	.0545	.0135
4	.0149	----	.0965	----	.0584	----
5	.0158	.0127	.0953	.0157	.055	.0135
6	.0157	.0126	.0953	.0166	.0915	.0135
7	.027	----	.0955	----	.094	----

TABLE 8.—TIN (PORT 3): FLUX CALCULATION

Size Range M	$\frac{d_{ave}}{14}$	$\left(\frac{d_{ave}}{14}\right)^3$	No	$\left(\frac{d_{ave}}{14}\right)^3$ No	V(d)	V_t
0-24.5	0.875	----	----	----		
24.5-31.5	2.0	8.0	23	184	.0042	0.7
31.5-38.5	2.5	15.65	38	594	.0132	1.0
38.5-45.5	3.0	27.0	310	8360	.1860	1.3
45.5-52.5	3.5	42.8	168	7200	.1602	1.7
52.5-59.5	4.0	64.0	285	18250	.4070	2.2
59.5-66.5	4.5	91.1	35	3180	.0709	2.6
66.5-73.5	5.0	125.0	43	5380	.1197	3.0
73.5-80.5	5.5	166.2	1	166	.0037	3.4
80.6-87.5	6.0	216.0	7	1510	.0336	3.8
			910	44824	.9984	

$$V(D) = \frac{\left(\frac{D_{ave}}{14}\right)^3 \text{No}}{\Sigma\left(\frac{D_{ave}}{14}\right)^3 \text{No}}$$

in which ω is an "average" rotational speed of the gas eddies in radians per second. For turbulent diffusion to be the mechanism of backmixing, the ratio of Eq. 21 should be nearly unity. To estimate ω, it was first assumed that the eddy size was one-fourth the height of the channel and that its velocity of rotation would be 10% of the mean velocity in the system.[4] Thus, an eddy 0.46 in. in diameter rotating at 5 ft per sec will have a frequency of 130 radians per sec.

Computation Procedure.—The experimental diffusion coefficient was computed as follows: First, an accurate velocity profile was established in the test section before the introduction of each test dust (Fig. 13). The profile was tested again with the dust in the stream. It is notable that the velocity profile did not change appreciably with the addition of particulate matter. This is probably due to the relatively low concentrations used.

With the establishment of the profile, seven equally spaced points were isokinetically sampled for particle concentration and recorded. The values of concentration were then plotted semi-logarithmically against distance from the wall. If a straight line was obtained as for sand (Fig. 10b), an expression for the concentration gradient $\left(\frac{dC}{dy}\right)$ was readily obtainable. In most cases, however, a linear plot had to be made (Fig. 14), and the gradient determined by drawing the tangent to the portion of the curve in question and measuring the slope. Needless to say, this method may lead to error but can be used conveniently if reasonable care is used in drawing.

The next step in determining the diffusion coefficient from Eq. 13 is that of determining a value of V_t C or flux. The magnitude of the terminal velocity, however, presents some difficulty because it varies as the particle size changes. It, therefore, becomes necessary, when a wide range of particle size exists, to determine the cumulative value of V_t C by a microscopic analysis.

BY MICROSCOPIC PARTICLE COUNT

Point 1 Flux $(V(d)V_tC)$	Point 2 Flux $(V(d)V_tC)$	Point 3 Flux $(V(d)V_tC)$	Point 4 Flux $(V(d)V_tC)$	Point 5 Flux $(V(d)V_tC)$	Point 6 Flux $(V(d)V_tC)$	Point 7 Flux $(V(d)V_tC)$
.0225	.00786	.002908	.00123	.000508	.000274	.000198
.1010	.03520	.013090	.00563	.002285	.001233	.000892
1.8500	.66250	.239500	.10100	.041750	.022600	.016280
2.0850	.72800	.271000	.11400	.047200	.025720	.018400
6.8500	2.39000	.886000	.37400	.149000	.083500	.060400
1.4100	.49200	.182000	.07690	.031800	.017200	.012420
2.7400	.91900	.33100	.13950	.059500	.032200	.023200
.0962	.03355	.012450	.00526	.002175	.001174	.000848
.9770	.34000	.126500	.05340	.022100	.011900	.008610
16.1117	5.60811	2.064448	0.87092	0.356318	0.195801	0.141248

Essentially the procedure is as follows. From each of the concentration samples taken at a given port, a microscopic particle size count is made of a representative portion of the dust collected. The counted particles are grouped into a size range and analyzed on a weight basis. The middle value of each group in the size range is called the average diameter d_{ave} . d_{ave} is then cubed and multiplied by the number of particles in the group. The product of $d^3{}_{ave}$ and the number of particles in the group is then divided by the sum of all $(d^3{}_{ave}N)$ and called V(d). V(d) times the concentration of the level in question times the terminal velocity of the average particle size would then represent the flux attributed to one group of particles in the size range.

The total flux for one level is obtained by adding the flux for each group in the range. A typical computation for tin appears in Tables 8 and 9. This analysis was not extended to the glass beads due to its rather narrow range of size. Once the flux was established, it was divided by the concentration gradient $\left(\frac{dC}{dy}\right)$ to give the observed value of diffusivity. A simpler approach to finding the flux is attained if the mass mean diameter $(\bar{d}m)$ is used in conjunction with the terminal velocity. This is particularly true if a narrow size range is encountered.

An example of a concentration profile is shown in Fig. 16. Experimental points show concentration values at various levels for a distance 13 ft downstream from the entry to the test section. Because the particles are relatively small (glass beads), the concentration gradient is a result of backmixing by turbulent diffusion against gravity. In curve B, the concentration profile is computed on the basis of gravity separation with laminar flow. If simple gravity settling occurred, the upper part of the test section, above a certain level, would be void of particulate matter. Repeated experiments have shown that such profiles (Curve A) are always obtained and an attempt has been made to show that turbulent diffusion is the mechanism responsible for concentration distributions of this type if the particles are small.

Comparison of Theoretical and Experimental Results.—In order to evaluate the effects of diffusivity, three dust samples were chosen for study. The dusts were subjected to the Weiss-Longwell criterion to determine the diffusivity ratios and the results obtained were as shown in Table 6. From the diffusivity ratios, little diffusion was expected from the tin and sand.

Table 7 is a summary of computed and observed diffusion coefficients. All samples taken for this test were made at the same distance downstream from the entrance section (port 3). A sample computation appears in Table 10.

It is to be noted that the glass beads follow the diffusion theory rather well. Theoretical values for positions one, four, and seven are deleted since the correction factor tends to make the values at position four (center line) equal to zero. Positions one and seven were not computed because a portion of the collecting tube is submerged in the boundary layer and the turbulent diffusion equations are not valid for that region.

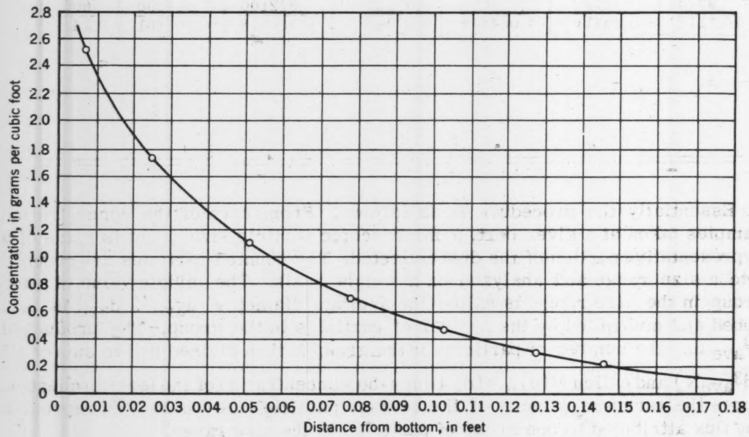

FIG. 15.—VARIATION OF CONCENTRATION WITH HEIGHT OF SAND

In the case of the sand particles, the observed diffusion coefficients are approximately six times as much as those computed by theory. This would indicate that backmixing primarily occurs because of some other mechanism.

The tin particles, having a low diffusion ratio compared to the diffusion of the gas, also exhibited an apparent diffusivity greater than that predicted by the theory.

An attempt was made to see if diffusion could be responsible for distributing the particles in the system according to size. It was felt that diffusion might be responsible for moving the smallest particles to the top of the duct. A microscopic analysis of tin and sand samples indicated that no significant difference occurred in the distribution of particle size in the vertical direction.

Analysis of Data Obtained.—Errors involved in diffusion computations may be numerous. The concentration gradient could not be obtained by graphical methods with any degree of precision. Wherever possible, (when the concentration plotted as a straight line on semi-log paper) the equation of the line was determined and the gradient established by taking the first derivative. Another error of considerable note is that of buildup and sloughing of particles in the system giving unusually high concentrations. This effect was reduced to a minimum by repeating each reading at least three times. Other errors may be incurred in weighing, microscopic counts, and positioning of sampling tubes.

Comparison with Other Studies.—While a direct comparison of data with other studies is not available, the diffusion theory used in this portion of the study is identical to that derived by H. A. Einstein,[13] F. ASCE, in his work with sediment transport in rivers.

Backmixing by Bouncing.—

Theory.—Irregularly shaped particles do not necessarily reflect when they strike a surface but bounce at some angle with a probability distribution about the average angle of reflection. Because a particle bouncing at a low angle has a certain probability of reflecting at a high angle, irregular bounce is a mechanism of backmixing.

TABLE 9.—COMPUTATION FOR TIN (PORT 3)

Point	C, in grams per cu ft	Flux[a] v_t C	$\frac{\partial C}{\partial y}$	Observed D	β	β u *	y x 10^{-3}	β u* y	D'
1	7.64	16.11	1030.	.0157	0.36	0.793	7.5	0.00595	----
2	2.67	5.61	175.0	.032	0.36	0.793	25.6	0.0203	.0135
3	0.992	2.06	37.8	.0545	0.36	0.793	51.0	0.0404	.0135
4	0.418	0.871	14.95	.0584	0.36	0.793	76.7	0.0607	----
5	0.173	0.356	6.5	.055	0.36	0.793	51.0	0.0404	.0135
6	0.0934	0.196	2.14	.0914	0.36	0.793	25.6	0.0203	.0135
7	0.0675	0.141	1.5	.094	0.36	0.793	17.58	0.00595	----

[a] As computed by microscopic particle count. $D' = \beta \, u^* y \left(\frac{D-y}{D}\right)$

A particle striking a collecting surface at an angle θ with a velocity v_i will be reflected at some angle $(\theta + \alpha)$ with a velocity E v_i. α is the angle of dispersion with a probability distribution about the average angle of reflection. E, the coefficient of restitution, is equal to or less than unity and represents the elasticity of the bounce. After rebound, the particle will have a y-velocity of E v_i sin $(\theta + \alpha)$ which will carry it a certain "stopping distance" away from the surface before the separating force, which is directed toward the surface, can decelerate the particle to a standstill and start it toward the surface once again.

To get an idea of the order of magnitude of backmixing by bounce, a simplified system is considered here. The approach angle is very small $(\theta = 0)$; the particle hits with the average velocity of the gas stream $(v_i = v_g)$, and the bounce is assumed to be completely elastic $(E = 1)$. The dispersion angle is distributed normally about the reflection angle, which is zero, with dispersion through the surface being reflected away from the surface, such that

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[13] "The Bed Load Function for Sediment Transport in Open Channel Flows," by H. A. Einstein, Tech. Bulletin No. 1026, U. S. Dept. of Agric., Washington, D.C., September, 1950.

$$\frac{2\,\delta}{\sqrt{\pi}} \exp\left(-\,\delta^2\,\alpha^2\right) d\alpha \quad \dots\dots\dots\dots\dots \quad (22)$$

represents the fraction of the total number of bouncing particles which bounce at an angle between α and $d\alpha$.

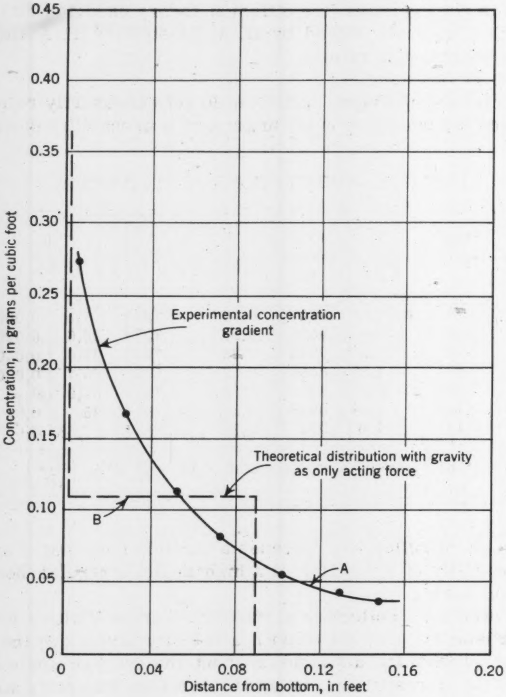

FIG. 16.—CONCENTRATION VARIANT–DEMONSTRATING BACKMIXING

The velocity away from the surface becomes

$$v_y = v_g \sin\alpha \approx v_g\,\alpha \quad \dots\dots\dots\dots\dots \quad (23)$$

in which α is an angle less than 15°. It is further assumed that the stopping distance of a particle bouncing at an angle is governed by Stoke's resistance law.

It is now assumed that at every y-distance above the surface, particles falling toward the surface are falling with their terminal velocity. This is certainly not true of particles whose bounce carry them only to this level, but the concentration at this level is made up primarily of particles "raining down" from much higher levels.

At equilibrium, a flux balance at a certain level reads

$$C\ v_t = \int_{\alpha}^{\infty} C_W\ v_t\ \frac{2\ \delta}{\sqrt{\pi}} \exp\left(-\ \delta^2\ \alpha^2\right) d\alpha \ \ldots\ldots\ldots \quad (24)$$

where the first term represents the flux toward the wall caused by the separating force and the integral term represents the flux away from the surface of those particles that have stopping distances greater than the y level in question. $C_W\ v_t$ is the flux of particles into the surface; $\frac{2\ \delta}{\sqrt{\pi}} \exp\left(-\ \delta^2\ \alpha^2\right) d\alpha$ is the fraction of particles bouncing at an angle α, reaching a stopping distance $S_s = y$, and $\delta = \frac{1}{\sigma_m}$. C_W is the concentration at the wall, v_t denotes the terminal velocity, and σ_m refers to the standard deviation in degrees.

TABLE 10.—CONCENTRATION GRADIENT AND DIFFUSION COMPUTATION OF GLASS BEADS

Point Port 1	C	$y \times 10^3$	v_t	$v_t\ C$	$\frac{\partial C}{\partial y}$	D' obs.	β	$\beta\ u^*$	$\beta\ u^*\ y$	$D_{eff.}$
1	0.1983	7.5	0.18	0.0377	3.25	0.0116	0.48	1.087	0.00816	----
2	0.158	25.6	0.18	0.030	1.995	0.0151	0.48	1.087	0.0278	0.0185
3	0.1135	51.0	0.18	0.0216	1.03	0.0210	0.48	1.087	0.0556	0.0186
4	0.0883	76.7	0.18	0.0168	0.547	0.0307	0.495	1.165	0.0845	----
5	.086	51.0	0.18	0.01635	0.257	0.0635	0.549	1.245	0.0634	0.0212
6	.0815	25.6	0.18	0.0155	0.1035	0.150	0.549	1.245	0.0318	0.0212
7	.0805	7.5	0.18	0.0153	0.0443	----	0.549	1.245	0.00923	----
Port 2										
1	0.117	7.5	0.18	0.0318	1.97	0.0161	0.398	0.884	0.00663	----
2	0.145	25.6	0.18	0.0261	1.52	0.0171	0.398	0.884	0.0226	0.0151
3	0.111	51.0	0.18	0.0200	1.16	0.0171	0.398	0.884	0.0450	0.0151
4	0.863	76.7	0.18	0.0162	0.799	0.0203	0.398	0.884	----	----
5	0.739	51.0	0.18	0.0133	0.606	0.0203	0.392	0.870	0.0444	0.0149
6	0.590	25.6	0.18	0.0106	0.54	0.0196	0.392	0.870	0.0223	0.0149
7	0.493	7.5	0.18	0.0086	0.503	0.0176	0.392	0.870	----	----
Port 3										
1	0.2705	7.5	0.18	0.0488	3.33	0.00147	0.392	0.877	0.00656	----
2	0.1575	25.6	0.18	0.0254	2.30	0.0124	0.392	0.877	0.0224	0.0149
3	0.115	51.0	0.18	0.0207	1.43	0.0145	0.392	0.877	0.0446	0.0150
4	0.0877	76.7	0.18	0.0158	1.122	0.0141	0.361	0.806	0.0616	----
5	0.0654	51.0	0.18	0.0118	0.783	0.0152	0.33	0.74	0.0378	0.0127
6	0.0465	25.6	0.18	0.00835	0.564	0.0148	0.33	0.74	0.0190	0.0126
7	0.0389	7.5	0.18	0.0070	0.259	0.027	0.33	0.74	0.0056	----

Rearranged, Eq. 24 gives

$$\frac{C}{C_w} = 1 - \mathrm{erf}\ (q)$$

$$= 1 - \frac{2}{\sqrt{\pi}} \left(q - \frac{q^3}{3 \cdot 1!} + \frac{q^5}{5 \cdot 2!} - \frac{q^7}{7 \cdot 3!} + \dots \right) \quad \dots\dots\dots\dots\dots \quad (25)$$

where $\mathrm{erf}\ (q) = \int_0^q e^{-q^2}\, dq$ (probability or error function) and

$$q = \alpha\, \delta \quad \dots\dots\dots\dots\dots\dots\dots \quad (26)$$

The effective diffusion coefficient becomes

$$D_{eff} = \frac{v_t\, C}{\frac{dC}{dy}} \quad \dots\dots\dots\dots\dots\dots\dots \quad (27)$$

Because α cannot be obtained explicitly in terms of y, the effective diffusion coefficient must be obtained by graphical differentiation of a plot of $C = C(y)$.

Computation Procedure.—Because reliable data on the dispersion angle of small particles with low approach angles could not be obtained, this information was computed from the concentration profiles in the following manner. From a point of known y-distance from the wall, the concentration value was obtained and placed into Eq. 25. The known y represented a related stopping distance to the angle of rebound α that was required to bounce a particle to that height. A similar procedure was followed for a second point. Two equations were now available with the unknowns C_w and q, where $q_1 = \alpha_1\, \delta$, and $q_2 = \alpha_2\, \delta$. By dividing the resulting equations, an expression for δ may be obtained for each type of particulate matter in question.

Once δ is established, Eq. 25 can then be used to establish the concentration at the wall and resulting concentrations at other levels. It is to be noted that δ is constant for a given material but α varies from level to level and is related to S_s, the stopping distance, as $S_s = S_s\ (\alpha)$. The relation between the stopping distance S_s and α, the rebound angle necessary to convey a particle to that height appears in Fig. 17. Sample computations appear in Appendix II.

Comparison of Theoretical and Experimental Results.—A comparison of theoretical and experimental concentration data appears on Table 11. With the exception of concentrations adjacent to the wall, the values observed and computed are almost identical. It must be borne in mind, however, that close agreement exists throughout the entire depth of suspended material (with the exception of that area near the wall), although only two points on the observed gradient have been used as match points to obtain a theoretical curve. Once "δ" has been established for a given material, however, only one concentration in the stream must be known in order to find the entire gradient. The comparison of diffusion coefficients, although not exact, definitely establishes an order of magnitude.

Analysis of Data Obtained.—The close correlation between observed and theoretical data as shown on Table 11 would indicate the validity of the pro-

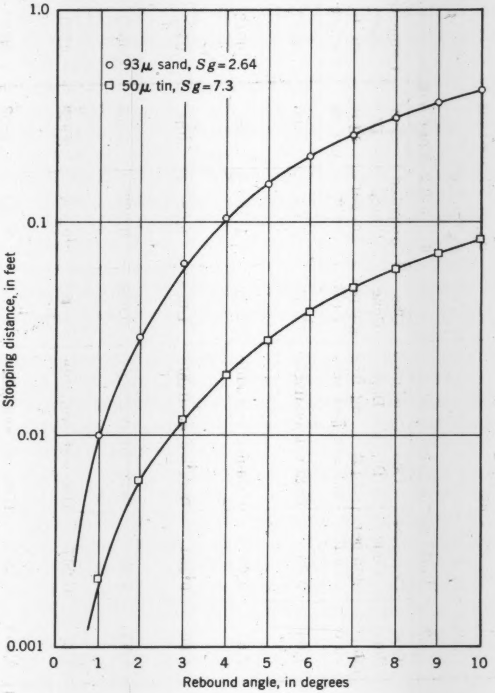

FIG. 17.—VERTICAL STOPPING DISTANCE FOR 50 FT PER SEC HORIZONTAL PARTICLE VELOCITY

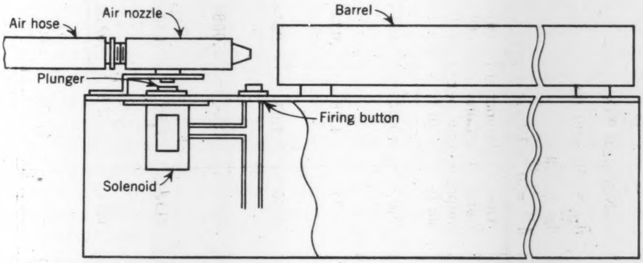
Air hose
Air nozzle
Barrel
Plunger
Firing button
Solenoid

FIG. 18.—AIR GUN

TABLE 11.—SUMMARY OF BOUNCE DATA

	ANGULAR TIN $\bar{d}_m$ = 50 microns Sg = 7.3 δ = 0.2825 per degree				ROUNDED TIN $\bar{d}_m$ = 50 microns Sg = 7.3 δ = 0.547 per degree				ANGULAR SAND $\bar{d}_m$ = 93 microns Sg = 2.64 δ = 0.275 per degree			
Y, in feet	Obs Conc. gm per cu ft	Computed Conc. gm per cu ft	Obs D' sq ft per sec	Computed D_{eff} sq ft per sec	Obs Conc. gm per cu ft	Computed Conc. gm per cu ft	Obs D' sq ft per sec	Computed D_{eff} sq ft per sec	Obs Conc. gm per cu ft	Computed Conc. gm per cu ft	Obs D' sq ft per sec	Computed D_{eff} sq ft per sec
0.0075	7.0	5.35	.0157	.0527	7.64	3.38	Data not available		2.54	2.63	.098	.0383
0.0256	4.37	4.38	.032	.092	2.62	2.07	Data not available		1.75	1.75	.089	.0512
0.054	3.35	3.33	.0545	.095	1.00	0.994	Data not available		1.099	1.08	.084	.0563
0.0767	2.5	2.56	.0584	.0901	0.437	0.43	Data not available		0.654	0.627	.0965	.0527
0.102	1.95	1.98	.055	.098	0.18	0.207	Data not available		0.398	0.397	.0953	.0553
0.1275	1.42	1.515	.0915	.101	0.093	0.0925	Data not available		0.259	0.219	.0953	.0503
0.1458	----	1.255	.094	.0968	0.067	0.507	Data not available		0.204	0.167	.0955	.0567

posed theory. Values of concentration near the wall that show a large difference might be due to; (1) errors in sampling near the wall, (2) the inability to determine the exact concentrations at the wall, and (3) the formation of a moving "bed load" at the wall.

In number one, perfect isokinetic sampling in the vicinity of the floor of the channel is almost impossible due to the large change in velocity with change of height. In actual procedure, the stream velocity that occurs at the center of the sampling probe was used thus leaving the lower half of the tube sampling at a rate much higher than conditions warrant due to a rapidly decreasing velocity as the wall of the channel was approached. At other sampling points above the floor, the velocity gradient is not so steep and an isokinetic condition is maintained fairly well.

The inability to determine exact concentrations at the wall is due to wall thickness of the sampling probe. When the probe was lowered to and resting on the floor, the wall thickness of the probe was the limit in the approach to the bottom.

The formation of a bed load in the laminar boundary layer immediately above the channel floor may also intensify differences between observed and computed values. If such a moving bed load existed, concentrations near the bottom would be considerably higher than the rest of the flow stream and, thus, are not accounted for in the bounce theory.

Of particular note, is the fact that the δ values for the angular materials were very close to each other. Because only two materials were tested, a conclusive statement cannot be made but the important significance is implied. On the other hand, the δ value for the rounded particles had a smaller dispersion, which is to be expected from a rounded particle.

Perfectly spherical particles might be expected to have an average rebound angle very close to zero but channel roughness may cause a significant dispersion.

Comparison with Other Studies.—With the exception of Bagnold,[4] who briefly deals with the height that large sand particles rebound from very rough surfaces, work of this nature is not cited in the available literature.

Special Tests Demonstrating Bounce Properties.—In order to gain some knowledge of bounce characteristics, a crude model test was devised. Essentially, the test consisted of shooting distorted ping-pong balls from an air gun at various angles and recording the rebound angles. The gun used is shown in Fig. 18. The distorted balls, which represented irregular particles, were made by boiling plastic balls slightly smaller than an ordinary ping-pong ball.

The gun was fired in different positions from horizontal to 40°. Bounce angles were obtained by markings, through carbon paper, at points of contact on the horizontal surface and a vertical backboard.

That the shapes representing the particles were of a random configuration was borne out by a statistical computation involving the Null Hypothesis.[14] Essentially, five representative shapes were fired about 50 times each from a constant gun angle and each shape showed a very high degree of correlation with each other although visual observation showed them to be of a completely different configuration.

When the data from the different gun-angles were assembled, it was found that the dispersion-angle varied with the incident-angle.

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[14] "A Simplified Guide to Statistics," by G. M. Smith, Rinehart and Co., Inc., New York, 1957, p. 55.

An attempt was made to see if any relation existed between positive deviations from the mean rebound-angle. The positive angles were chosen since a negative rebound-angle could not exist in the test system. It was found that the standard deviation varied from approximately 10° for a gun angle of 40° to 5½° for a gun angle of 10° (Fig. 21).

Although no correlation existed for various approach angles, it was found that the mean rebound angle was approximately equal to the approach for incident angles less than 20° and that the total distribution for any given gun angle was very close to a normal distribution (Fig. 20).

It would appear that information gathered from this model study could easily be adapted for use in a field unit where the separating force is considerably larger than simple gravity. The cyclone, for example, might induce forces causing a 20 micron particle to approach the collecting surface at 45°.

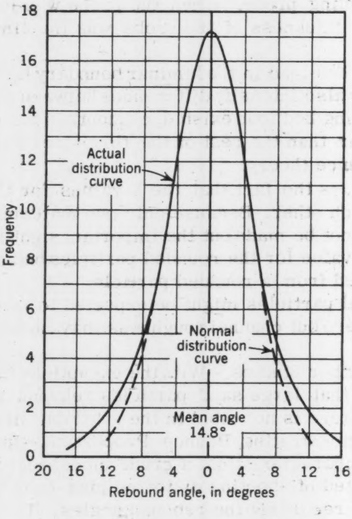

FIG. 19.—TYPICAL REBOUND ANGLE DISTRIBUTION

CONCLUSIONS

Theory predicts quite well the shear stresses and velocities that accompany pickup from a wall or particle layer. Of particular note is the fact that the velocities or shear stresses can be predicted from a few simple bench top experiments on the dust in question, and a knowledge of flow conditions in the system.

Backmixing by turbulent diffusion occurs for small particles if the proper turbulence conditions exist. If the Weiss-Longwell criterion for diffusion produces a diffusion ratio close to unity, backmixing by turbulent diffusion can be expected. The concentration gradient can be predicted with a fair degree of accuracy if the velocity profile and one point of concentration are known.

Bouncing of the particles in a gas suspension would seem to govern the condition of backmixing in the case of larger particles. A concentration field, based on the normal distribution of bounce angle and model data, can be predicted rather accurately if one point of concentration is known.

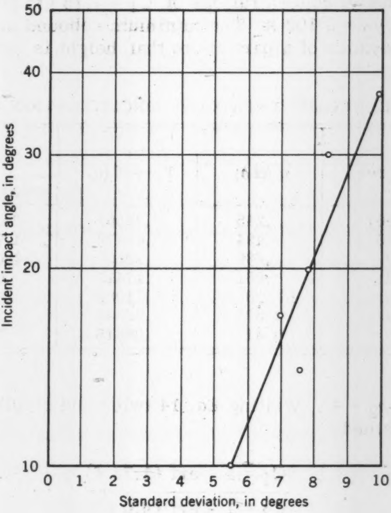

FIG. 20.—DISPERSION RELATION WITH INCIDENT ANGLE BASED ON MODEL STUDY

Although all of the proposed theories for this work have been developed for simple cases, modifications can be made for their application to more complex systems.

ACKNOWLEDGMENTS

This investigation is part of research under Research Grant S-19(c) undertaken at the Pennsylvania State University under the sponsorship of the United States Public Health Service (USPHS).

The work was also supported (in part) by a traineeship (AT57-201) from the USPHS.

APPENDIX I.—SAMPLE COMPUTATION OF SAND CONCENTRATION BY BOUNCE

In the determination of δ from observed data, best agreement with theory was found when the concentrations used in Eq. 25 were those somewhere near midstream whose values fell on, or very close to, the straight section of the line of best fit of log C versus y.

From log C versus y, concentrations of $C_1 = 1.75$ gm per cu ft at $y = .0256$ ft and $C_2 = 0.398$ at $y_2 = 0.102$ ft. The minimum rebound angle α which is necessary for the conveyance of a particle to that height is obtained from Fig. 17 and $\alpha_1 = 1.75°$, and $\alpha_2 = 4°$. Writing Eq. 14 twice and dividing one by the other the following is obtained

TABLE 12.—COMPUTATION OF CONCENTRATION PROFILE

y, in feet	α degrees	q ($\alpha\,\delta$)	1 - erf (q)	Concentration	
				Computed	Observed
.0075	.867	.245	.72898	2.63	2.54
.0256	1.75	.494	.48479	1.75	1.75
.051	2.6	.735	.2986	1.08	1.099
.0767	3.4	.961	.17413	.627	.654
.102	4.0	1.13	.11033	.397	.398
.1275	4.7	1.327	.06056	.219	.259
.1458	5.0	1.41	.04615	.167	.204

$$\frac{C_1 = 1 - \mathrm{erf}\ (1.75\ \delta)}{C_2 = 1 - \mathrm{erf}\ (4.0\ \delta)} \quad \ldots\ldots\ldots\ldots\ (28)$$

or

$$4.4 = \frac{1 - \mathrm{erf}\ (1.75\ \delta)}{1 - \mathrm{erf}\ (4.0\ \delta)} \quad \ldots\ldots\ldots\ldots\ (29)$$

By assuming a value of delta (δ), the error function can be integrated. The assumed values of delta are continued until the right hand side of the equation is equal to the left. Conveniently, the error function or probability integral is computed in most tables of integrals and a fast solution is obtained.

The solution for δ in Eq. 1 yields $\delta = 0.2825$, and $1 - \mathrm{erf}\ (1.75\ \delta)$ is equal to 0.48479. The concentration at the wall can now be found by substitution in the orginial equation (Eq. 24).

$$\frac{C}{C_w} = 1 - \mathrm{erf}\ (\alpha\ \delta) \quad \ldots\ldots\ldots\ldots\ (30)$$

$$C_w = \frac{1.75}{0.48479} = 3.63 \quad \ldots\ldots\ldots\ldots \quad (31)$$

The computation of the concentration profile is best shown in tabular form and is as shown in Table 12.

APPENDIX II.—NOTATION

The following letter symbols have been adopted for use in this paper:

A = Constant equal to half the channel depth;

b = Constant;

C = Concentration, in grams per cubic foot;

C_w = Concentration at wall, in grams per cubic foot;

C_D = Coefficient of drag;

D^1 = Actual diffusion coefficient, in square feet per second;

D_{eff} = Effective diffusion coefficient, in square feet per second;

d_p = Diameter of particle, in feet, except where otherwise noted;

E = Coefficient of restitution;

erf = Error function;

f^* = External friction factor;

F_D = Aerodynamic force (grams or pounds force);

F_R = Sliding and rolling friction force (grams or pounds force);

Pt = Point, refers to height in channel;

q = $\alpha\,\delta$;

R = Reynold's Number;

S_g = Specific gravity;

S_s = Stopping distance, in feet;

u = Velocity in x direction, in feet per second;

u^* = Friction velocity, in feet per second;

v_o = Effective velocity, in feet per second;

$V(d)$ = Mass fraction;

v_t = Terminal velocity, in feet per second;

v_i = Incident velocity, in feet per second;

v = Velocity gradient at wall;

W = Weight of particle, in pounds;

y = Distance measured from bottom wall of channel, in feet;

α = Dispersion angle in degrees;

β = Constant, same as b;

γ = Specific weight;

ΔP = Pressure difference $(P_2 - P_1)$ (inches of water);

δ^1 = Boundary layer thickness, in feet;

δ = Constant equal to $1/\sigma_m$ (degree^{-1});

μ' = Turbulent viscosity, in pound-seconds per square foot;

μ_g = Gas viscosity, in pound-seconds per square foot;

ν = Kinematic viscosity, in square feet per second;

ω = Angular velocity, in radians per second;

ϕ = Friction angle in degrees;

ψ = Internal friction factor, or grain interlock;

ρ_p = Particle density, in pound-seconds squared per foot.^{4}

ρ_g = Gas density, in pound-seconds squared per foot.^{4}

σ_m = Standard deviation in degrees;

τ = Shear stress, in pounds per square foot; and

τ_o = Shear stress at wall, in pounds per square foot.

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Paper No. 3145

USE OF ALGAE IN REMOVING PHOSPHORUS FROM SEWAGE

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and J. C. Pluntze[3]

SYNOPSIS

Eutrophication of receiving waters by nutrient rich wastes may be controlled by removal of phosphorus. Phosphorus can be removed from sewage by biological and chemical means. Either approach is aimed at converting soluble phosphorus to easily recovered insoluble particulate matter. An attempt was made during the period from 1957 to 1960 to exploit the metabolic activities of algae in removing phosphorus from treated sewage. A tertiary stage treatment process designed to strip phosphorus from solution through use of algal photosynthesis was developed in the laboratory. The process was subsequently studied in both laboratory and field scale pilot plants, and the basic principles of this process are outlined. Research leading to process development is briefly described, and process performance is evaluated and analyzed.

INTRODUCTION

Excessive enrichment or eutrophication of receiving waters by nutrient-rich wastes is emerging as a major water pollution problem. It has been recognized for some time that ordinary domestic sewage is a rich source of the nutrients

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Note.—Published essentially as printed here, in September, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2605. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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required by phytoplankton. Experience has shown that the degree of eutrophication and hence the severity of subsequent water quality problems is largely dependent on the supply of inorganic nitrogen and phosphorus.

Attempts to control or prevent excessive plankton blooms have usually involved some manner of chemical treatment such as periodic application of $CuSO_4$, or diversion of nutrient-rich wastes to less sensitive or less valuable receiving waters, or a combination of these measures. Both of these control procedures have their limitations. Cost of the best available algicides precludes their use for continuous control of most eutrophic waters. Furthermore, the effect of most algicides is only temporary and does not get at the real cause of the problem. Diversion would appear to be a more effective and expedient course of action, but here, too, there are some serious shortcomings. As more and more drainage basins become involved, as now seems to be the case, the long range value of diversion grows increasingly doubtful. Moreover the economics of diversion are difficult to evaluate. They are frequently lost in the argument that there is no alternative solution.

From time to time interest has developed in some method of waste treatment which would remove offending fertilizing elements before discharge. Sawyer[4] has shown that removal of phosphorus offers a practical and effective way of controlling algal growths in most natural waters. Phosphorus removal may be accomplished by biological and by chemical means. Either approach is aimed at converting soluble inorganic phosphorus into recoverable insoluble matter. Of the two, chemical coagulation has received the greatest attention, and several promising but costly chemical treatment methods have been proposed.[5,6,7] Little has been done to date with biological treatment.

The concept of employing algae as a means of removing nutrients from domestic sewage is examined in this paper. Subsequent development of a treatment process employing a photosynthetic unit operation will be described. Laboratory and field scale pilot plant studies were made on removal of phosphorus from the effluent of an activated sludge plant.

THEORETICAL CONSIDERATIONS

Chemical Treatment Processes.—It is possible by means of chemical coagulation to reduce the soluble phosphorus content of sewage to concentrations as low as 0.10 to 0.50 mg per l. Lime, iron salts, filter alum and copper sulfate have been investigated.[4,5,6,7] Chemical requirements have been found to vary with the sewage being treated, with pH, and with the nature of the inorganic phosphate being removed. In general, optimum chemical doses have been found to range from about 150 mg per l to 400 mg per l or more.

The work of Lea, Rohlich, and Katz at the University of Wisconsin, Madison, Wis., indicates that alum and iron salts are equally effective in removing phosphorus. Optimum coagulant dose for secondary sewage treatment plant effluent was found to be in the vicinity of 200 mg per l. The effectiveness of various chemicals in reducing phosphate solubility is shown[5] in Table 1. Comparable phosphate reductions have been reported for lime doses ranging from 300 to 700 mg per l of $Ca(OH)_2$.[4,6]

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[4] "Some New Aspects of Phosphates in Relation to Lake Fertilization," by C. N. Sawyer, Sewage and Industrial Wastes, Vol. 24, No. 708, 1953.

[5] "Removal of Phosphates from Treated Sewage," by W. L. Lea, G. A. Rohlich, and W. J. Katz, Sewage and Industrial Wastes, Vol. 26, No. 261, 1954.

[6] "Removal of Phosphorus from Sewage Plant Effluent with Lime," by R. Owen, Sewage and Industrial Wastes, Vol. 25, No. 548, 1953.

[7] "Phosphates in Sewage and Sludge Treatment II Effect on Coagulation, Clarification and Sludge Volume," by W. Rudolfs, Sewage Works Journal, Vol. 19, No. 178, 1947.

The mechanisms of phosphate removal by chemical coagulation is not too well understood. Theoretically, phosphorus may be removed from solution through precipitation as an insoluble salt or by adsorption upon some insoluble solid phase. Available experimental evidence indicates that both mechanisms may be operative, particularly at low residual phosphorus concentrations. In the case of lime coagulation it appears that the principle mechanism is that of precipitation as insoluble calcium phosphate salts. With iron salts and alum, adsorption upon hydrated oxide floc-particles appears to play a major role. Pilot plant data indicate that floc settling properties may require much lower clarifier overflow rates than commonly employed in sewage treatment.

Cost appears to be the major limitation of all chemical coagulation processes studied to date. Based on available information the cost of chemicals alone would range from about $25 to $100 per million gallons of sewage treated for a 90% or more reduction in soluble phosphorus. Obviously the degree of phosphorus reduction will have some bearing on chemical requirements and attendant costs.

TABLE 1.—COMPARISON OF COAGULANT EFFICIENCY AT DOSES OF 200 mg PER l

Coagulant	Conc. Soluble P mg per l		Removal Soluble P, in percent
	Initial	Residual	
Alum	5.37	0.07	98.7
Ferrous Sulfate	5.67	0.06	99.0
Ferric Sulfate	6.02	0.05	99.0
Copper Sulfate	6.16	0.24	96.1

Biological Treatment Processes.—The concept of removing nutrients biologically can hardly be considered as new or unique. In any actively growing system nutrient materials are continually extracted from the environment through conversion to cell tissue. Rate of nutrient removal, other things being equal, is a function of the rate of cell tissue synthesis, whereas the amount of nutrient reduction is determined by cell tissue composition and the mineral content of the medium. Growth rates vary greatly with type of organism and with the species. The mixed microbial culture provided by the activated sludge process would appear to be the most effective biological system in terms of removal rate.

Examination of the data presented[8] in Table 2 indicates that assimilation of 1 mg per l of phosphorus by algae would be accompanied by metabolism of 33 mg per l to 78 mg per l of carbon and 1 mg per l to 12 mg per l, or more, of nitrogen. Similar considerations also apply to bacteria.

Ordinary domestic sewage does not provide a balanced diet. Both carbon and nitrogen are deficient with respect to the amount of phosphorus normally present. In the case of activated sludge, organic carbon is usually limiting. Adequate amounts of carbon are normally available to algae in the form of alkalinity. There also is evidence that atmospheric nitrogen fixation might serve as a significant source of nitrogen in large scale algal cultures.[4] Viewed in terms of nutritional requirements algae appear to offer the most easily exploited biological system for extracting phosphorus from sewage.

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[8] "Algal Culture from Laboratory to Pilot Plant," by R. W. Krauss, Carnegie Inst. of Washington, Publication 600, Chapter 8, Table 3, 1953, p. 90.

Conventional biological sewage treatment should, of course, remove some phosphorus. With a BOD to phosphorus requirement in the range of 100 to 300: 1 for a typical aerobic reactor and a settled raw sewage BOD in the range of 100 mg per l to 200 mg per l it becomes obvious that phosphorus reductions during the course of complete biological treatment would, on the average, be limited to approximately 1 mg per l. Owen,[6] in an investigation of sewage treatment plant performance in Minnesota, found phosphorus removal ranged from an average 2% for primary treatment plants to an average 23% for plants employing biological treatment. This was equivalent to approximately 1 mg per l to 2 mg per l of phosphorus. Rudolfs[9] reported phosphorus reductions during the course of biological treatment running as high as 75% to 90%! Analysis of sewage treatment plants in the Seattle, Wash. area disclosed reductions ranging from 15% to 40%, which was equivalent to 0.80 mg per l to 2.0 mg per l of phosphorus.

TABLE 2.—COMPOSITION OF FRESH WATER ALGAE

Element	Per cent Total Dry Weight
Carbon	49.5 - 70.2
Oxygen	17.4 - 33.2
Hydrogen	6.6 - 10.2
Nitrogen	11.4 - 11.0
Phosphorus	0.9 - 1.5
Sulfur	.09
Magnesium	0.3 - 1.5
Calcium	0.0 - 1.5
Potassium	0.0 - 1.4

The Use of Algae in Sewage Treatment.—The general role of algae in sewage treatment has received considerable attention in recent years.[10,11,12] Unfortunately very little has been reported regarding nutrient reductions stemming from algal activity. Available information indicates phosphorus reductions ranging from 10% to 90% or more. Performance appears to be erratic and unpredictable. Considerable difficulty has been experienced in harvesting algal cell tissue. This difficulty, coupled with slow growth rate, would account for some of the wide fluctuations noted in the mineral composition of many oxidation pond effluents.

Recovery and Re-use of Algae.—A relatively simple process was envisioned whereby algae would be employed as a means of removing phosphorus from sewage. In essence, the process would consist of a growth cell or lagoon followed by a separation device for recovering algal cell tissue. Algae would be recycled for re-use or wasted to a sludge-holding lagoon according to need. Sewage would be mixed with actively growing algae in a lagoon or growth cell. The principal problem, at the outset, appeared to be one of recovering cell tissue.

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[9] "Phosphates in Sewage and Sludge Treatment I Quantities of Phosphates," by W. Rudolfs, Sewage Works Journal, Vol. 19, No. 43, 1947.

[10] "Algal Symbosis in Oxidation Ponds—II Growth Characteristics of Chlorellapyrenoidosa Cultured in Sewage," by W. J. Oswald, et. al., Sewage and Industrial Wastes, Vol. 25, No. 25, 1953.

[11] "Algal Symbiosis in Oxidation Ponds—III Photosynthetic Oxygenation," Ibid., Vol. 25, 1953, p. 692.

[12] "Development of Design Criteria for Waste Stabilization Ponds," Final Report to the AEC, Civ. Engrg. Dept., Univ. of Texas, March 1, 1957.

A number of harvesting operations have been investigated. All have been found wanting in some aspect, generally in terms of cost, often in terms of efficiency. From the standpoint of complete performance and economy some type of screening device appears to be the most promising. Screening-performance would obviously be related to the nature of the algal culture. It was reasoned, however, that if some readily recovered alga could be established through the simple mechanism of recovery and re-use it could, in turn, be made to predominate. Thus, the process culture would be made to adjust to the most readily utilized population. Much of the early portion of this research was influenced by these concepts.

RESULTS

During the course of this investigation, a number of basic concepts came under study. A brief summary of some of the more significant findings are presented with emphasis on the questions of usable algal cultures, observed phosphorus removal, $PO_4^{\equiv}$ solubility, photosynthetic pH shift, and pilot plant performance.

Algal Cultures.—A search was made for usable algae with particular interest in large filamentous species. Cultures were grown in aerated 2 l glass tubes at several temperatures ranging from 10° C to 25° C. Light intensity was maintained at 400 to 500 ft-c. Various mixtures of lake water and raw or treated sewage were employed as seed and culture media. During most of the laboratory phase of this research an inorganic synthetic sewage was used; to approximate the composition of secondary sewage treatment plant effluents it was designed in the Seattle area land; its composition is shown in Table 3.

Several common fresh water algae were grown. However, except for Chlorella and Scenedesmus, most types died after a brief period of growth. A large filamentous alga, subsequently identified as Stigleoclonium stagnatile, was recovered from the rock of a biological filter in the area and successfully cultured. This alga when grown under aeration developed into setteable floc particles resembling activated sludge. It subsequently became the subject of a large part of the laboratory phase of this investigation. Photomicrographs of Stigleoclonium stagnatile are shown in Fig. 1. Fig. 1(a) shows floc-like colonies which developed in aerated cultures. The magnification is 220 times. Fig. 1(b) shows a view of the individual organism. The magnification is 520 times. Its growth characteristics and nitrogen and phosphorus content are shown in Tables 4 and 5. By way of comparison, the growth rates of a number of other algae are shown in Table 5.

Observed Phosphorus Removals.—The rate at which an algal culture may be expected to extract phosphorus from solution should be a function of growth rate, cell tissue concentration, and the phosphorus content of the cell tissue. Computed theoretical metabolic uptake rates for cell tissue containing 2% phosphorus (dry weight) are shown in Fig. 2. Light intensity will determine cell tissue concentration. Temperature, diet and species will regulate k. In the

TABLE 3.—SYNTHETIC SEWAGE COMPOSITION

Substance (1)	Source (2)	Amount Added to Tap Water as Ion or Element, in mg per l (3)	Amount Present in Tap Water,[a] in mg per l (4)
N	$Ca(NO_3)_2$	10	—
	$K\,NO_3$	20	—
	or		
	NH_4Cl	30	—
P	KH_2PO_4	10	—
Na	$NaSiO_2$	5.0	1.91
	$NaCl$	39.0	
K	KNO_3	55.6	0.32
	KH_2PO_4	12.2	
Ca	$CaCl_2$	varies	7.11
	$Ca(NO_3)_2$	14.3	
Mg	$MgSO_4$	4.8	0.70
Fe	$FeSO_4$	0.20	0.05
Cl^-	$CaCl_2$	varies	1.15
	$NaCl$	30.0	
HCO_3^-	$NaHCO_3$	100.0	25.1
SO_4	$MgSO_4$	15.0	2.05
SiO_2	$NaSiO_2$	15.0[b]	8.0

[a] Based on analytical data obtained from city of Seattle Engineering Department for years 1951 - 1955.

[b] Reduced to 1.5 mg per l for pilot plant operation.

TABLE 4.—EFFECT OF TEMPERATURE AND CULTURE MEDIA ON GROWTH RATE STIGLEOCLONIUM STAGNATILE[a]

Temperature, °C (1)	Synthetic Sewage Employing NO_3-N (2)	Synthetic Sewage Employing NH_3-N (3)	Secondary STP Effluent (4)
	k $days^{-1}$[c]	k day^{-1}[c]	k $days^{-1}$[c]
10	0.165	0.140	0.170
15	0.188	0.179	0.215
20	0.252	0.131	0.131[b]

[a] pH varied from 8.3 - 9.5, illumination was at 400 ft c.

[b] Scenedesmus appeared and began to predominate after approximately 15 days.

[c] k computed from $N_t = N_o e^{kt}$.

Seattle area algal concentration ranged from 25 to 50 mg per l in 4 ft deep lagoons. The mean k was 0.30 days.$^{-1}$ The soluble phosphorus content of most laboratory cultures was observed to decrease at a rate considerably in excess of that predicted by biological uptake alone. Response to repeated heavy doses of phosphate was most interesting. In general some 80% to 90% of the $PO_4^{\equiv}$ added was removed from solution within 2 hr as shown in Fig. 3. Aerated Stigleoclonium cultures exhibited a remarkable capacity for coagulating and adsorbing ortho phosphate. Photosynthetic response was not visably impaired by repeated use of cell tissue. Obviously more than metabolic uptake was involved.

TABLE 5.—COMPARISON OF ALGAL GROWTH RATES[a]

Organism (1)	k^a days^{-1} (2)	Generation Time days (3)	Generation Time minutes (4)
Anabaena cylindrica	0.74	0.94	1350
Chlorella pryenoidosa	0.48 - 2.0	0.35 - 1.4	500 - 2070
Chlorella vulgaris	0.67 - 1.1	0.63 - 1.0	600 - 1500
Euglena gracilis var. bacillaris	0.79	0.71	1020
Scenedesmus quadricauda	2.0	0.35	500
Scenedesmus costulatus	0.28 - 1.08	0.64 - 1.4	920 - 2070

[a] $L_{ne} \frac{Nt}{No} = k\,t$

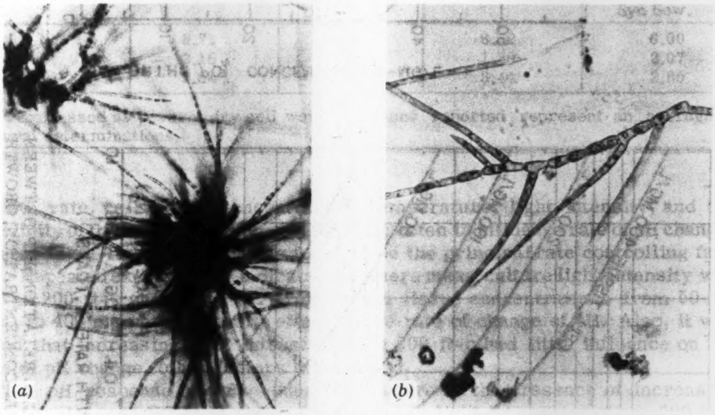


FIG. 1.—PHOTOMICROGRAPHS OF STIGLOECLONIUM STAGNATILE

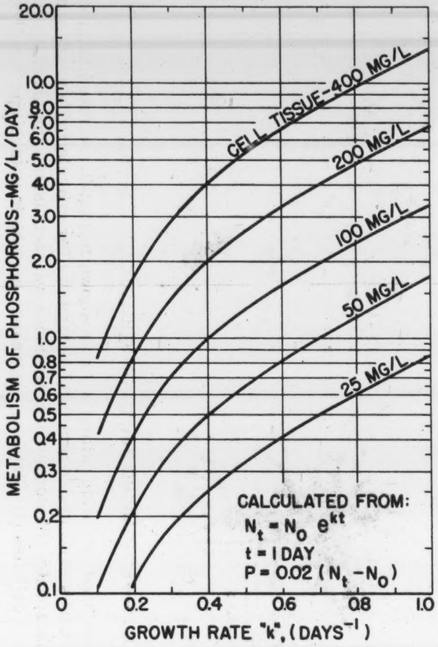


FIG. 2.—THEORETICAL RELATIONSHIP BETWEEN CELL TISSUE CONCENTRATION, GROWTH RATE AND METABOLIC CONVERSION OF PHOSPHORUS BY ALGAE

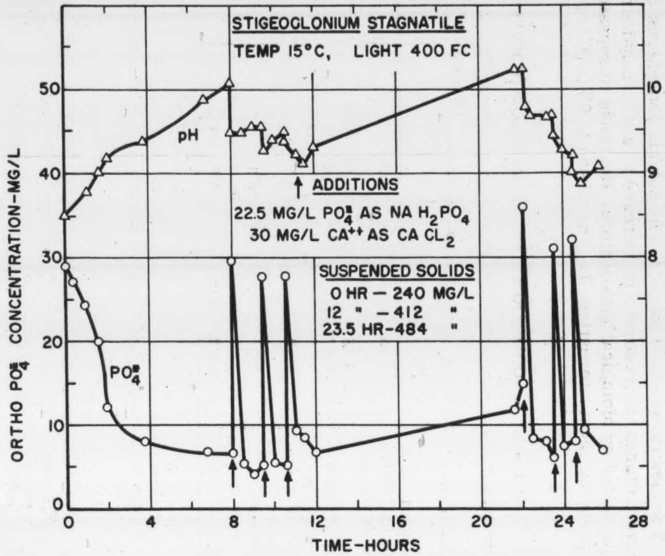


FIG. 3.—RESPONSE OF BATCH FED ALGAL CULTURES TO REPEATED DOSES OF $PO_4^{\equiv}$

Examination of culture characteristics disclosed that phosphorus removal was closely related to pH. It appeared that coagulation and adsorption may have played a major role. Accordingly, the next phase of the investigation was concerned with the physical chemical behavior of $PO_4^{\equiv}$ and particularly those factors affecting solubility.

Factors Affecting $PO_4^{\equiv}$ Solubility.—Orthophosphate may combine with a number of substances ordinarily present in sewage to form relatively insoluble complexes under suitable conditions. Calcium ion concentration and pH were found to be the principal controlling factors in determining $PO_4^{\equiv}$ solubility. The relationship between $PO_4^{\equiv}$ solubility, pH and Ca^{++} concentration is shown in Fig. 4. Calcium ion concentration and pH were adjusted by means of $CaCl_2$, $Ca(OH)_2$ and NaOH. Samples of synthetic sewage and sewage treatment plant effluent were flocculated for 15 min; settled supernatant was filtered and the filtrate analyzed for soluble phosphorous. Ammonia, iron, and magnesium in amounts normally encountered in domestic sewage did not exercise any discernable effect on $PO_4^{\equiv}$ solubility.

Photosynthetic pH Shift.—A study was made of the role of algae in adjusting pH. This adjustment of pH is presumable related to consumption of CO_2 and an attendant shift in the carbonate-bicarbonate ion balance. Such factors as growth rate, cell tissue concentration, temperature, light intensity, and the alkalinity of the culture medium would be expected to influence rate of pH change. Interestingly, light intensity was found to be the principal rate controlling factor. Repeated experiments indicate that where mean culture light intensity was 100 to 200 ft-c and above variation in cell tissue concentrations from 50 mg per l to 400 mg per l had little effect on the rate of change of pH. Also, it was noted that increasing light intensity above 200 ft-c had little influence on the rate of pH change, other factors being equal.

TABLE 6.—PHOSPHOROUS AND NITROGEN CONTENT OF STIGLEOCLONIUM STAGNATILE HARVESTED FROM VARIOUS CULTURE MEDIA[a]

Constituent	Synthetic Sewage		Secondary STP	1/2 STP Eff.
	NO_3 - N	NH_3 - N	Effluent	1/2 NO_3 - N Syn Sew.
N	5.71	6.59	6.52	6.00
P	2.16	1.81	1.89	2.07
N/P	2.64	3.63	3.44	2.89

[a] Expressed as percent dry cell weight. Values reported represent an average of several determinations.

The pH response of a Scenedesmus culture in the presence of increasing concentrations of HCO_3 is shown in Fig. 5. At light intensities above 200 ft-c Scenedesmus, Chlorella and Stigleoclonium caused a rapid increase in the pH of raw and treated sewages. Alkalinity in amounts generally encountered in most sewages had little affect on pH response. These data are typical where light was not a limiting factor. Generally speaking, the pH level of secondary plant effluent would increase to 9.0-9.1 in 4 to 6 hr. Much higher pH levels were ultimately reached as these data indicate; however, there was usually a lag period of 6 to 12 hr preceeding the second increase in pH level.

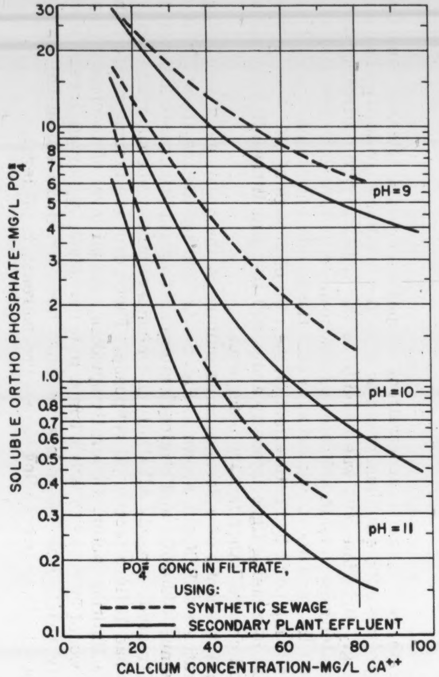


FIG. 4.—INFLUENCE OF pH AND CALCIUM CONCENTRATION ON ORTHOPHOSPHATE SOLUBILITY

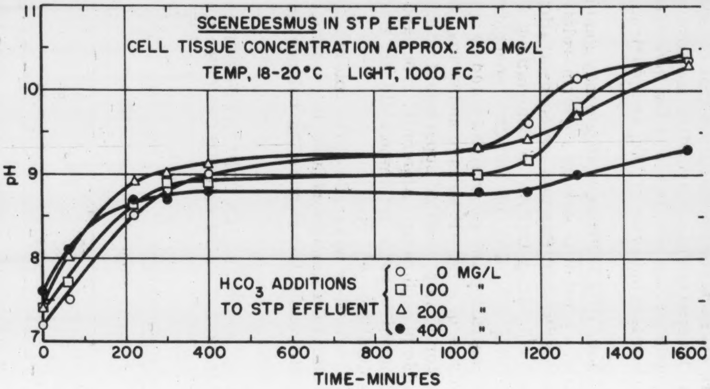


FIG. 5.—INFLUENCE OF HCO_3^- CONCENTRATION ON PHOTOSYNTHETIC pH SHIFT

Data presented in Table 7 describe photosynthetically induced pH changes observed in open basins during field pilot plant studies. Mean light intensities through the algal culture were on the order of 10 to 20 ft-c. Liquid depth was approximately 3 1/2 ft. These data are typical of raw sewage lagoons and oxidation ponds in the Seattle area during the summer and early fall.

Laboratory Pilot Plant Studies.—The forementioned findings were subsequently translated into a laboratory scale pilot plant. This pilot plant consisted of an illuminated contact unit, a clarifier, and a basin or lagoon for regeneration of algal cell tissue. A flow diagram of the laboratory pilot plant is shown in Fig. 6.

The process was operated on both synthetic sewage and on effluent from an activated sludge plant in the area. Contact time, recirculation ratio and the effect of lagoon or regeneration basin operation were studied. Typical performance data are shown in Fig. 7. Actual $PO_4^{\equiv}$ removals have been compared with theoretical values computed from the data presented in Fig. 4.

TABLE 7.—PHOTOSYNTHETIC pH CHANGES IN LAGOON CELLS - AUTUMN 1958

Lagoon Cell	Time, in days								Remarks
	1	2	3	5	7	10	12	15	
1	7.7	7.6	8.4	8.7	8.5	9.4	9.5	10.1	Intermittent aeration plus artificial illuminations @400 ft-c during dark hours
2	8.0	7.7	8.4	8.8	8.9	9.2	9.2	9.6	Int. aeration: no artificial illum.
3	8.1	8.0	8.6	9.1	9.3	9.7	9.7	10.4	No.aeration; no art. illumination
4	8.1	7.5	8.2	10.4	—	—	—	—	Int. aeration. Air off after day 6
Time of Sampling	1:30 pm	11 am	11 am	11:30 am	10:30 am	3 pm	11:30 am	12 am	All lagoon cells filled with secondary treatment plant effluent and seeded with equal volume of algal culture
Weather	Cloudy	Cloudy	Pt. Cldy	Rain	Sunny	Pt. Cldy	Sunny	Sunny	
Temperature °C	—	—	14	—	12	11	10	11	
Suspended Solids mg per l	—	28	33	36	36	62	62	70	

The significance of Ca^{++} concentration and of pH is obvious. It also was found that sludge recirculation ratios up to 3: 1 were beneficial and that little was to be gained by increasing holdup time in the contact unit beyond 4 hours in the presence of 200 ft-c or more of light.

When the process was operated on secondary sewage treatment plant effluent, the Stigleoclonium culture became contaminated with Chlorella and Scenedesmus. At temperatures above 20° C, Chlorella and Scenedesmus tended to predominate; the Stigleoclonium was simply overgrown. This gave rise to some misgivings at first until it was observed that settling characteristics

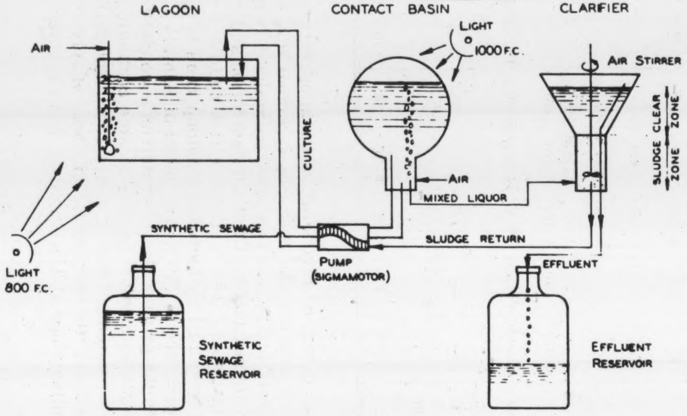
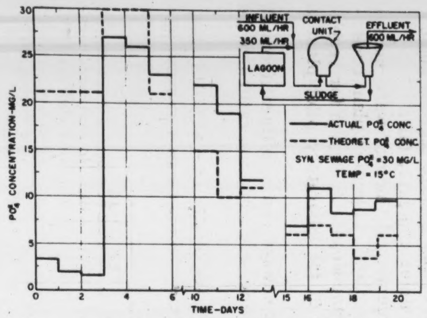
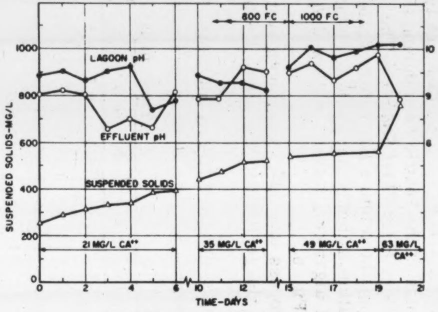

LAGOON
CONTACT BASIN
CLARFIER
AIR
LIGHT
1000 F.C.
AIR STIRRER
CULTURE
AIR
MIXED LIQUOR
SLUDGE ZONE
CLEAR ZONE
SYNTHETIC SEWAGE
SLUDGE RETURN
PUMP
(SIGMAMOTOR)
EFFLUENT
LIGHT
800 F.C.
SYNTHETIC
SEWAGE
RESERVOIR
EFFLUENT
RESERVOIR

FIG. 6.—FLOW DIAGRAM OF LABORATORY SCALE PILOT PLANT



(a) PHOSPHOROUS REMOVALS IN LABORATORY PILOT PLANT



(b) LABORATORY PILOT PLANT OPERATING CONDITIONS

FIG. 7.—TYPICAL LABORATORY PILOT PLANT PERFORMANCE

varied moderately with algal species owing to the coagulation effect of the insoluble phosphate salts produced at high pH levels.

Field Pilot Plant Studies.—A field scale pilot plant was constructed employing the findings of the laboratory studies as a basis for design. The plant was operated on the effluent from an activated sludge sewage treatment plant. The contact unit and clarifier each provided two hours retention time and an overflow rate of 360 g per sq ft day at design flow of 10 gpm. The lagoon phase of the operation was designed to provide five days retention of the algal culture sludge anticipated in the process. Artifical illumination of the contact tank and the lagoon cells was provided by means of industrial fluorescent fixtures. The overall dimensions of the pilot plant were 32 ft by 16 ft by 4 ft deep. A flow diagram of the pilot plant is shown in Fig. 8. Fig. 9 shows the plant. The influent pump is in the foreground, the illuminated contact tank at the left and the lagoon cells to the right.

Inability to achieve a mean light intensity above 100 ft-c markedly inhibited rate of change of pH. The effect of this was to render the contact unit inoperative. Where adequate detention time was available, such as in the lagoon cells, suitable pH levels were subsequently realized as shown in Table 7.

When it became apparent that photosynthesis alone would not provide the desired degree of pH adjustment, attention was directed toward supplemental use of lime. The effect of various lime doses was first studied in the laboratory. Lime doses giving the desired pH change or $PO_4^{\equiv}$ reduction were then applied in the field. Performance employing both lime and algae taken from lagoon cells is shown in Fig. 10. At light intensities prevailing in the Seattle area during the fall, rapid photosynthetic pH adjustment was not possible. Phosphorus removal was due entirely to the influence of lime. Actual $PO_4^{\equiv}$ residuals have been compared with theoretical values computed from measured pH and Ca^{++} concentration. The discrepancy between theory and actual performance appears to be related to sedimentation tank performance. Performance was in good agreement with theory since the maximum difference noted was equivalent to approximately 10% of the influent $PO_4^{\equiv}$ content.

ANALYSIS

An attempt was made to exploit the metabolic activities of algae in removing nutrients from treated sewage. A tertiary stage treatment process designed to strip phosphorus from solution through use of algal photosynthesis was developed in the laboratory. The process subsequently was studied both on a laboratory and on a field-scale pilot plant basis. Based on these studies it appears that the removal of inorganic phosphorus from solution by algae is the result of both metabolic uptake and of physical-chemical adsorption and coagulation. Adsorption and coagulation appeared to play the major role where rapid removal of large concentrations of $PO_4^{\equiv}$ was involved. The relative significance of metabolic uptake naturally depends on environmental conditions and upon time available for growth. In either case, photosynthetic activity governed rate and extent of $PO_4^{\equiv}$ reduction.

Controlling Variables.—Both Ca^{++} concentration and pH combine to act as the principal regulator of phosphate solubility. Photosynthesis in turn serves to regulate pH. Thus any factor which serves to control algal growth rate will

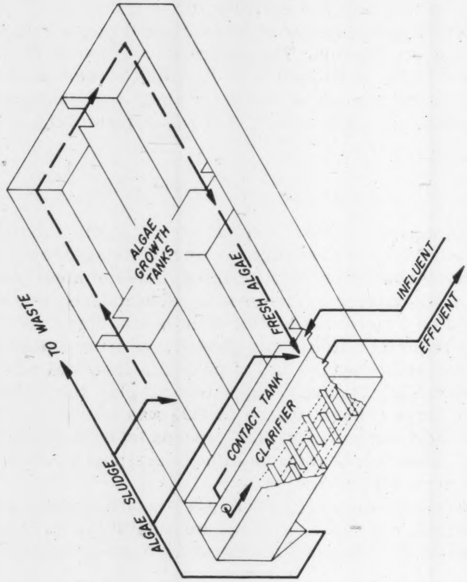

ALGAE SLUDGE
TO WASTE
ALGAE GROWTH TANKS
FRESH ALGAE
CONTACT TANK
CLARIFIER
INFLUENT
EFFLUENT

FIG. 8.—FLOW DIAGRAM OF FIELD PILOT PLANT

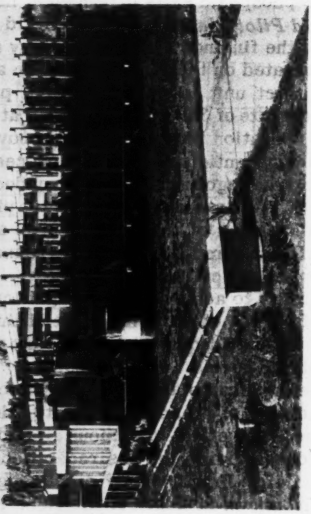


FIG. 9.—FIELD SCALE PILOT PLANT

also regulate process performance. Several potentially controlling factors immediately come to mind including temperature, algal species, chemical constitution of the sewage being treated, and light intensity. Of these, light intensity was found to be the principal controlling factor. Where light was not limiting, the process appeared to be functionally sound, and was affected but little by other variables.

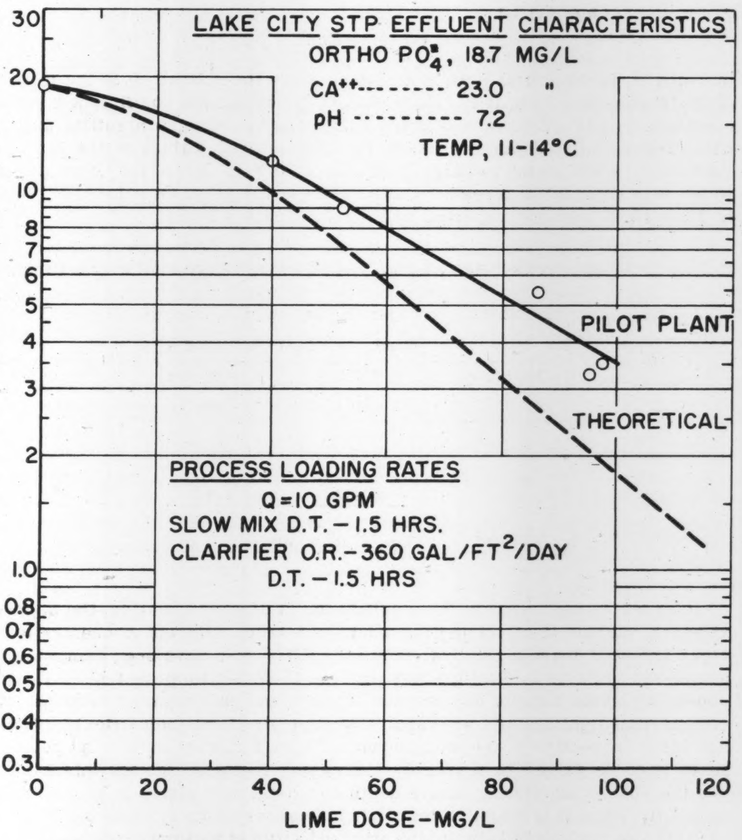


FIG. 10.—COMPARISON OF PILOT PLANT PERFORMANCE WITH THEORETICAL PHOSPHATE SOLUBILITIES

Light Requirements.—Most investigators have found optimum light intensities for algal cultures to lie in the range of 200 to 400 ft-c. Increasing light intensity beyond this value generally has little beneficial influence. Intensities above 1000 to 2000 ft-c often tend to inhibit photosynthesis. The results of this research suggest that adequate algal response as measured in terms of rate of change of pH was obtained at light intensities above 100 ft-c. It also appears that rate of change of pH at light intensities above 100 ft-c varies little as algal cell tissue concentrations are varied from 30 mg per l to 400 mg per l.

Optimum light requirements are easily determined experimentally in the laboratory. Unfortunately the problem of maintaining such conditions on a large scale is more difficult. The severity of the lighting problem is determined entirely by the light transmitting properties of the algal culture. The light transmission or adsorption characteristics of an algal culture may be described by the Beer-Lambert Law.

$$I_t = I_o e^{-Ecd} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which I_o is the initial light intensity entering the culture, I_t is the intensity of the transmitted light at any depth d, E is the extinction coefficient in square centimeters per milligram, c is the concentration of algae in milligrams per liter, and d is the depth expressed in centimeters. Values of the extinction coefficient were found to range from 2.0×10^{-3} to 3.9×10^{-3} cm^2 per mg, which is in substantial agreement with values reported by Oswald[13] (1.0×10^{-3} to 2.0×10^{-3}) and that reported by Tamiya[14] (3.8×10^{-3}).

TABLE 8.—RELATION[a] BETWEEN I_o, ALGAE CONCENTRATION, AND DEPTH at I_t = 100 ft-c

Concentration Algal Cell Tissue mg per l (1)	Depth - cm for Corresponding I_o			
	1000 ft-c (2)	2000 ft-c (3)	5000 ft-c (4)	10,000 ft-c (5)
50	23	30	39	46
100	11.5	15	19.5	23
200	5.8	7.5	9.8	11.5
400	2.9	3.8	4.9	5.8

[a] I_t computed from $I_t = I_o e^{-Ecd}$; $E = 2 \times 10^{-3} \frac{cm^2}{mg}$.

In order to demonstrate the significance of light attenuation, the distances at which various light intensities would penetrate different concentrations of algae and still leave a residual intensity of 100 ft-c have been computed, and are shown in Table 8. Sunlight may provide 1000 ft-c to more than 10,000 ft-c, depending on the time of day, season of year, and geographical location. Most commercial lighting units are capable of supplying light intensities of 2000 ft-c and less. Inspection of the data shown in Table 8 discloses that (a) relatively little is to be gained by markedly increasing incident lighting intensity, and (b) the photosynthetically active depth of most algal culture is generally less than 1 ft. When it is recalled that light intensities much in excess of 1000 ft-c to 2000 ft-c may actually be inhibitory, the futility of attempting to provide adequate illumination solely through increasing initial intensity, becomes all the more apparent.

Thus, in order to maintain maximum photosynthetic activity on a large scale it would be necessary to either restrict depths to a maximum value on the order of one foot, or to have sources of illumination located throughout the culture mass spaced, in general, less than 2 ft apart.

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[13] "Photosynthesis in Sewage Treatment," by W. J. Oswald and H. B. Gotaas, Transactions, ASCE, Vol. 122, 1957.

[14] "Algal Culture from Laboratory to Pilot Plant," Publication 600, Carnegie Inst. of Washington, Washington D. C., 1953.

Most lagoons and oxidation ponds are constructed to operate with liquid depths of 3 ft to 5 ft. Obviously, only a portion of the liquid depth may be considered as a zone of active photosynthesis. As a practical matter, where thorough mixing of pond volume takes place, the effect of employing depths greater than that involved in photosynthesis is roughly equivalent to illuminating the entire culture at a proportionally lesser intensity. The net photosynthetic response is thus reduced, and a longer contact time must be provided to achieve a given degree of pH adjustment. For example, it was found that 12 hr contact at 200 ft-c was approximately equivalent to 10 to 12 days holdup in a 3 1/2 ft deep lagoon under field conditions prevailing in the Seattle area during the fall.

Economic Evaluation.—When artificial illumination is employed, the cost of photosynthetic pH adjustment is determined by power requirements and by space or volume needs. A finite culture volume will be effectively illuminated by each light element and any portion of the culture lying outside this volume or at some greater distance will receive inadequate amounts of light. If the light intensity and power input of the light source together with the light transmitting properties of the culture are known, it is possible to express light requirements in terms of power per unit culture volume. This in turn may be translated into actual lighting cost if the illumination period is known. The cost of illuminating cultures by means of commercially available fluorescent elements has been computed and is shown in Fig. 11. Cost of electrical energy was taken at \$.01 per kw-hr. Power requirements were based on high voltage fluorescent elements and a culture extinction coefficient of 2×10^{-3} cm^2 per mg.

The cost of employing sunlight is determined by the cost of providing the necessary detention time. Detention times ranging from 2 to 10 or more days may be required to reach pH levels of 9.5 and above. It is contemplated that construction similar to that employed in oxidation ponds and lagoons would be used in this case. The cost of photosynthetically adjusting pH by means of sunlight, has been computed (Fig. 11) based on employing lagoons having an average depth of 4 ft, a unit volume cost of \$0.05 per cu ft, an expected life of 20 yr, and response equal to that observed in these studies (Table 7).

The cost of adjusting pH by means of $Ca\ (OH)_2$ is also shown in Fig. 11. Lime requirements were computed from titration curves obtained for sewage treatment plant effluents throughout the Seattle area. The cost of lime was taken at \$0.02 per lb. Inspection of these cost data suggest that the use of presently available means of artifical illumination is not economically feasible. Furthermore, it would appear that even under the most favorable of circumstances, natural illumination would be no more economical than lime. Of course, where adequate volume already exists in the form of oxidation ponds or sewage lagoons the question then becomes one of determining the cost of adapting and employing what is already available.

As mentioned earlier, the field scale pilot plant was operated employing both algae and lime. The cost of removing different amounts of $PO_4^{\equiv}$ employing algae only and algae in conjunction with various lime doses is shown in Fig. 12. Costs shown are for power and chemicals only. These costs are bases on employing artifical illumination together with sunlight to achieve photosynthetic pH adjustment.

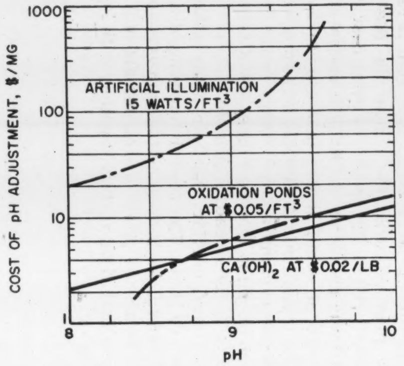

FIG. 11.—ECONOMIC COMPARISON OF VARIOUS METHODS OF ADJUSTING pH.

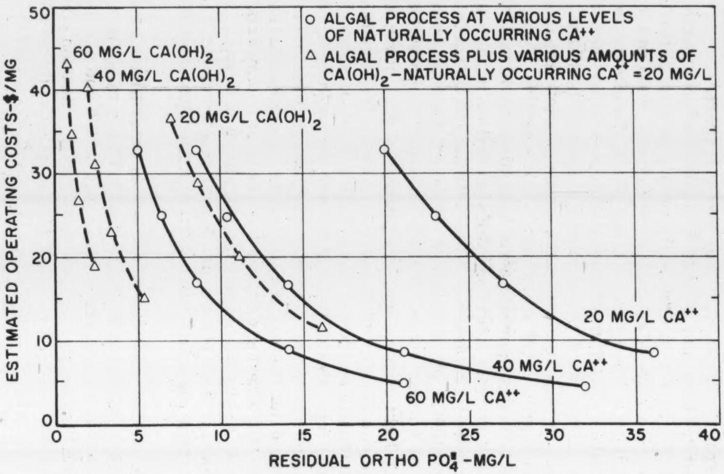

FIG. 12.—COMPARISON OF TREATMENT COSTS EMPLOYING HIGH RATE PROCESS

It should be noted that the concentration of naturally occurring calcium has a marked influence on treatment cost. This is readily understood when one recalls the relationship between pH, Ca^{++} concentration, and $PO_4^{\equiv}$ solubility shown in Fig. 4. Obviously, the mineral content of a community's carriage water may significantly alter the cost of biologically removing $PO_4^{\equiv}$, other things being equal.

The Use of Oxidation Ponds.—Theoretically, an oxidation pond should be capable of very high efficiencies of phosphorus removal. In order to realize this potential active photosynthesis and/or high pH conditions must prevail for some time prior to discharge. Where biological fixation is the principal mode of phosphorus removal, efficiency will be a function of detention time, growth rate, and cell tissue concentration.

During field pilot plant studies algal cell tissue concentrations in lagoon units having a depth of 3 ft to 4 ft varied from 25 mg per l to 50 mg per l. Average growth rate was equivalent to a k of 0.30 day^{-1}. Thus, in order to biologically extract 5 mg per l of P (equivalent to 80% to 90% reduction for most sewages) it appears that lagoon retention times on the order of 14 days to 28 days would be required. Theoretical retention time requirements for any other set of circumstances can be computed from the data presented in Fig. 2. These considerations suggest that detention times in excess of those commonly employed in most oxidation ponds may be necessary where a high degree of phosphorus removal is a treatment objective.

SUMMARY AND CONCLUSIONS

The possibility of employing algae as a means of biologically removing $PO_4^{\equiv}$ from domestic sewage was studied in the laboratory, and in the field, on a pilot plant scale. Early phases of this work were aimed at developing a process intended for use as a tertiary treatment stage wherein algae served as the sole means of $PO_4^{\equiv}$ removal. Under carefully controlled laboratory conditions, such a process was found to be functionally sound. Inability to maintain adequate light intensities under field scale conditions gave rise to relatively high treatment costs. The process as originally conceived was subsequently modified so as to include both the use of lime and the effect of natural illumination. The more significant aspects of this research are summarized as follows.

1. In the presence of adequate amounts of light it is possible to realize rapid biological extraction of $PO_4^{\equiv}$. For example, in a laboratory pilot plant orthophosphate concentration was reduced from 20 or more mg per l to less than 5 mg per l in less than 4 hr.
2. Adsorption and coagulation appear to play the major role where rapid removal of large amounts of phosphate is involved. Metabolic conversion is the principal mechanism of removal under the more leisurely conditions prevailing in oxidation ponds and sewage lagoons.
3. Three algae, Chlorella, Scenedesmus, and Stigleoclonium stagnatile, were grown in raw and treated sewages. Each was capable of rapidly increasing culture pH when illuminated at light intensities above 100 ft-c. Stigleoclonium grew in floc like particles resembling activated sludge.
4. Repeated use of cell tissue markedly improved subsidence properties without noticably impairing photosynthetic response. Settleability appeared to be influenced by the formation of insoluble $PO_4^{\equiv}$ compounds at elevated pH. Thus it appears that in addition to the prospects of removing $PO_4^{\equiv}$ there is also

the possibility that the photosynthetic pH shift may be employed as a means of enhancing the recovering of algae by sedimentation.

5. Light intensity was found to be the principal controlling variable. Under normal field conditions adequate light intensities seldom prevail in an algal culture at depths greater than 1 foot. The use of deeper ponds, as is common practice today, serves in effect to decrease the net mean illumination in proportion to the ratio of light to dark volumes.

ACKNOWLEDGMENTS

The initial portions of this research including the laboratory pilot plant studies were supported by the University of Washington Engineering Experiment Station. The city of Seattle provided funds for construction of the field scale pilot plant.

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Paper No. 3146

DESIGN PRINCIPLES FOR UNDERGROUND SALT CAVITIES

By Shosei Serata[1] and Earnest Gloyna,[2] M. ASCE

SYNOPSIS

Theoretical equilibrium relationships as substantiated by experimental studies are presented here for use in the design of salt cavities. Also included are studies on reduction of cavity volume, development of the plastic zone, and stress redistribution around the cavity as functions of cavity depth, strength of salt, and physico-chemical effects of the waste.

INTRODUCTION

The ultimate disposal of radioactive waste in quantity has become one of the most important problems of this age. The object of this paper is to present design principles for the construction of salt cavities as containers for high-level radioactive waste.

The theoretical investigations of spherical and cylindrical cavities included analysis of elastic stress, thermal stress, development of the plastic zone, reduction of cavity volume and the resulting stress redistribution around the cavities. The theoretical analyses have demonstrated that the structural equilibrium relations of a salt cavity can be established in relation to cavity temperature, structural loading condition, and cavity volume reduction.

Note.—Published essentially as printed here, in May, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2468. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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In the laboratory, the fundamental properties of salt as well as the various factors affecting these properties were investigated. By using a model salt cavity, the volume reduction in connection with the development of the plastic zone was studied. Measurements of creep in the Grand Saline salt mine were made and the results seem to agree with the theoretical and experimental conclusions.

ELASTIC STRESS ANALYSIS

Depending on the geometry, the stability of a salt cavity can be studied using the theory of elasticity when the depth of the cavity does not exceed a value of approximately 500 ft to 1,000 ft. The stress condition around a cavity can be mathematically determined for cavities of simple forms like a sphere, cylinder, or elliptical tunnel. For cavities of irregular form, the theory of photoelasticity can be successfully applied.[3] By using both theories, the effect of depth, size, shape, number, and geometry of cavity opening upon the elastic stress distribution are determined.

The initial stress naturally existing in any underground formation before an opening is made is primarily determined by two factors, the overburden load and the Poisson's ratio of the formation, as follows:

$$\sigma_y = \frac{\mu}{1 - \mu} \sigma_z \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which ν is Poisson's ratio, σ_y denotes the lateral stress, and σ_z is the vertical stress or overburden load.

Experimental determination of Poisson's ratio is necessary to determine natural stress distributions in salt. The results of both the laboratory experiments and the calculations on seismic data are in agreement. The value of Poisson's ratio is approximately 0.5 at a depth greater than 300 ft. The stress distribution in the salt therefore approximates that found under hydrostatic load.

Therefore, due to the overburden load, cavities created at a depth of 300 ft or above will be subject to a non-hydrostatic external stress condition; cavities below a depth of 300 ft will be subject to a hydrostatic stress condition. If some horizontal tectonic pressure is existing in the formation, this tectonic pressure should be superimposed over the above-mentioned stress conditions as illustrated in Fig. 1.

By creation of a cavity in an initially stressed salt formation, maximum and minimum stresses and maximum shearing stress always appear on the boundary around the cavity opening. The magnitude of these critical stresses depends on the form of the cavity, as illustrated in Fig. 2. In the diagram, the stress distribution around circular and rectangular openings is shown for the three fundamental initial stress conditions, hydrostatic, non-hydrostatic, and unidirectional.[4]

As seen in Fig. 2, the stress distribution is affected by the form of the opening as well as by the initial stress condition. However, no tensile stress oc-

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[3] "Design by Photoelasticity," by R. B. Heywood, Campman and Hall, Ltd., London, 1952.

[4] "Stress Around Mine Openings in Some Simple Geological Structures," by R. D. Caudle and G. B. Clark, Engrg. Experiment Sta. Bulletin No. 430, Univ. of Illinois, Vol. 52, No. 69, May, 1955.

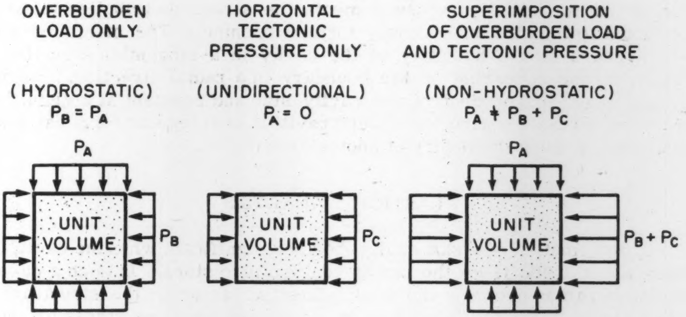
OVERBURDEN
LOAD ONLY
HORIZONTAL
TECTONIC
PRESSURE ONLY
SUPERIMPOSITION
OF OVERBURDEN LOAD
AND TECTONIC PRESSURE
(HYDROSTATIC)
$P_B = P_A$
(UNIDIRECTIONAL)
$P_A = 0$
(NON-HYDROSTATIC)
$P_A \neq P_B + P_C$
P_A
UNIT VOLUME
P_B
UNIT VOLUME
P_C
P_A
UNIT VOLUME
$P_B + P_C$

FIG. 1.—INITIAL STRESS CONDITIONS IN SALT FORMATION

FIG. I INITIAL STRESS CONDITIONS IN SALT FORMATION.

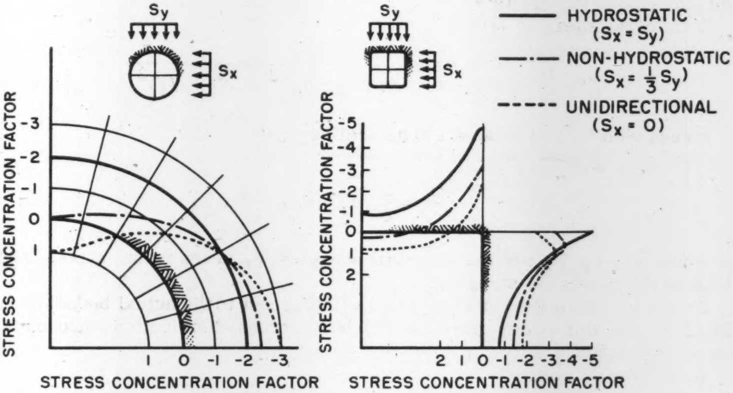

FIG. 2.—DISTRIBUTION OF CRITICAL STRESS AROUND CIRCULAR AND SQUARE TUNNEL OPENINGS

curs around the cavity unless the ratio of the major to minor initial principal stresses exceeds three. In reality, therefore, significant tensile stress is not probable around any form of salt cavity.

The effect of cavity size on stress distribution can be illustrated by the theoretical analysis of stress distribution around a spherical cavity, as plotted in Fig. 3. In an initially stressed media the stress distribution is directly proportional to the size of the newly formed opening. The maximum stress always appears on the boundary of the cavity in a tangential direction, the minimum stress appearing on the boundary in a radial direction. However, the stresses are independent of the cavity size and constant at a given cavity depth. The stress distribution around cavities of irregular form can also be investigated by using the theory of photoelasticity.

PLASTICITY ANALYSIS

Creep deformation of a salt cavity becomes significant when the cavity depth exceeds about 1,000 ft or the cavity temperature rises. In such a case, the plasticity of salt is probably the most influential factor on the structural stability of the salt cavity. Whenever the maximum shearing stress around the cavity exceeds a certain value, the salt starts to creep rather than fracture. Because of this creep, the salt cavity will never collapse under an increased loading condition. Addition of thermal stress merely produces a further extension of the plastic zone.

The development of the plastic zone in spherical and cylindrical cavities can be mathematically analyzed by introduction of the octahedral shearing stress (Eq. 2) into the following differential equation defining the stress equilibrium around the cavity:[5]

Octahedral shearing stress:

$$\tau_o = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \quad \ldots\ldots\ldots (2)$$

Stress equilibrium around a spherical cavity:

$$\frac{d\sigma_r}{dr} + \frac{2}{r}(\sigma_r - \sigma_t) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which σ_1, σ_2, σ_3 are the principal stresses, σ_r is the radial stress, and σ_t denotes the tangential stresses.

Solution of the differential equation with respect to the actual boundary condition of the salt cavity gives the following stress-distribution equation for a spherical cavity:

In the plastic zone:

$$\left.\begin{aligned} \sigma_r &= -p_i - 2\,\sigma_o \ln\frac{r}{a} \\ \sigma_t &= -p_i - 2\,\sigma_o\left(\frac{1}{2} + \ln\frac{r}{a}\right) \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (4a)$$

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[5] "Plasticity," by A. Nadai, McGraw-Hill Book Co., Inc., New York, 1931.

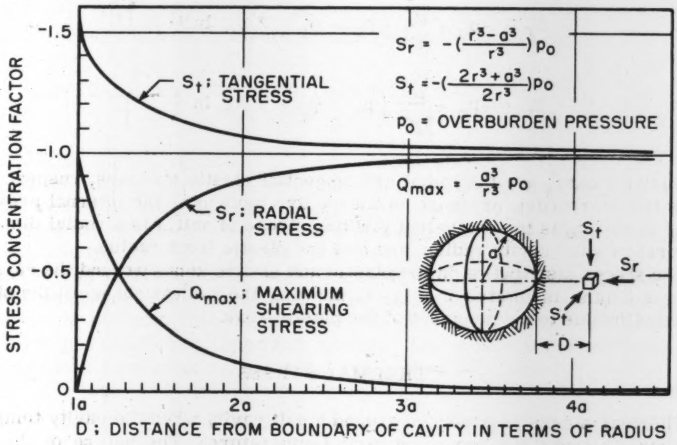

FIG. 3.—STRESS CONCENTRATION AROUND SPHERICAL CAVITY

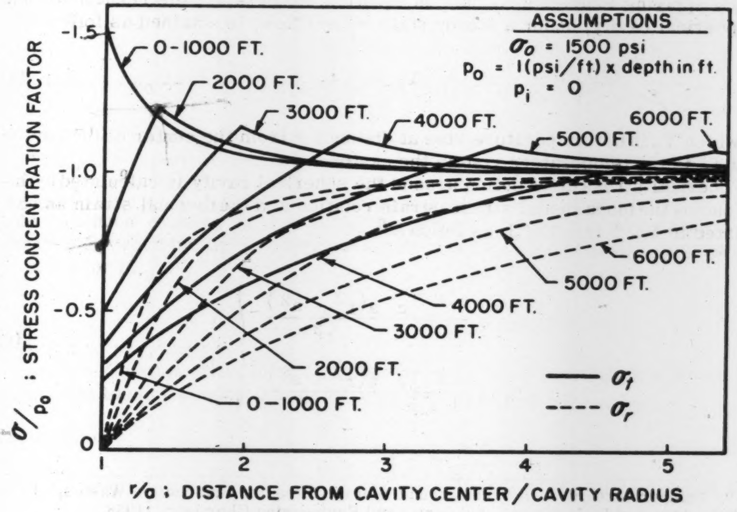

FIG. 4.—STRESS DISTRIBUTION IN PLASTIC AND ELASTIC ZONES AROUND SPHERICAL SALT CAVITY AT VARIOUS DEPTHS

In the elastic zone:

$$\left.\begin{aligned} S_r &= -p_o + \frac{\rho^3}{r^3}\left(p_o - p_i - 2\,\sigma_o \ln\frac{\rho}{a}\right) \\ S_t &= -p_o - \frac{\rho^3}{2r^3}\left(p_o - p_i - 2\,\sigma_o \ln\frac{\rho}{a}\right) \end{aligned}\right\} \quad \ldots\ldots\ldots (4b)$$

in which S_r and S_t are the radial and tangential elastic stresses, respectively; P_o is the overburden pressure of the cavity, p_i denotes the internal pressure of the cavity, σ_o is the equivalent yielding stress of salt, r is a radial distance, a represents the cavity radius, and ρ is the plastic front radius.

The stress distribution in the plastic and elastic zones around the cavity at various depths is plotted in Fig. 4, showing the radical change of the stress distribution due to development of the plastic zone.

THERMAL STRESS

Thermal stress is produced around a salt cavity when the cavity temperature rises above the surrounding earth temperature. The nature of the temperature rise in the cavity depends on three major factors: the nature and condition of the waste, the geometry of the cavity, and the thermal property of the salt.[6]

By applying Fourier's conduction equation, temperature distribution around a spherical cavity, under a steady state of heat flow, is obtained as follows:[7]

$$T_r = \frac{a}{r}\,T_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which T_r is the temperature rise at distance r from the center of the cavity, and T_c is the temperature rise in the cavity.

Thermal stress distribution around the spherical cavity is calculated by introducing the fundamental stress-strain relation and the thermal strain as calculated in Eq. 5 into Eq. 3, as follows:[8]

$$\left.\begin{aligned} S_r &= \frac{\alpha\,E\,T_c}{1-\mu}\;\frac{a\left(r^2 - a^2\right)}{r^3} \\ S_t &= \frac{\alpha\,E\,T_c}{1-\mu}\;\frac{a\left(r^2 + a^2\right)}{2\,r^3} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (6)$$

---

[6] "Temperature Rise in Underground Storage Sites for Radioactive Wastes," by R. S. Schechter and E. F. Gloyna, Industrial and Engineering Chemistry, 1958.

[7] "Heat Conduction with Engineering and Geological Applications," by L. R. Ingersoll, O. J. Zobel, and A. C. Ingersoll, McGraw-Hill Book Co., Inc., New York, 1948.

[8] "Theory of Elasticity," by S. Timenshenko and J. N. Goodier, McGraw-Hill Book Co., Inc., New York, 1951.

The actual thermal stress distribution due to elevated temperature around a spherical salt cavity can be calculated by introducing the proper values of the three coefficients for the salt, namely, Young's modulus, Poisson's ratio and the coefficient of linear expansion into Eq. 6. In Fig. 5, the distribution is plotted by using these coefficients determined in the laboratory.

DEVELOPMENT OF STRUCTURAL EQUILIBRIUM EQUATIONS FOR DESIGNING CAVITY

To design an underground salt cavity for the storage of reactor fuel waste, the relation among cavity temperature rise, structural loading, and cavity volume reduction must be established first. Such a relation is obtained by correlating the effects of elastic stress, thermal stress, development of plastic zone, and the thermal expansion of the salt.

The stress distribution around a cavity with elevated temperature can be determined by superimposing the thermal stress over the plastic and elastic stresses. It is found that the stress distribution equation in the plastic zone (Eq. 4a) is not affected by temperature rise. The stress distribution equation for the elastic zone around a spherical cavity then becomes:

$$S_r = -p_o + \frac{\rho^3}{r^3}\left[p_o - p_i - 2\,\sigma_o \ln\frac{\rho}{a} + \frac{\alpha E T_c}{1-\mu}\,\frac{a(\rho^2 - a^2)}{\rho^3}\right] - \frac{\alpha E T_c}{1-\mu}\,\frac{a(r^2 - a^2)}{r^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7a)$$

$$S_t = -p_o + \frac{\rho^3}{2\,r^3}\left[p_o - p_i - 2\,\sigma_o \ln\frac{\rho}{a} + \frac{\alpha E T_c}{1-\mu}\,\frac{a(\rho^2 - a^2)}{\rho^3}\right] - \frac{\alpha E T_c}{1-\mu}\,\frac{a(\rho^2 - a^2)}{\rho^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7b)$$

The condition existing at the borderline between the plastic and elastic zone can be expressed by Eq. 8, and by substitution of Eq. 7 into Eq. 8, it is possible to solve for ρ.

$$S_r - S_t = \sigma_o \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Thus, the equation describing the plastic front is obtained in relation to the structural loading condition and the cavity temperature rise.

$$\frac{p_o - p_i}{\sigma_o} = 2\ \ln\frac{\rho}{a} + \frac{2}{3} - \frac{2}{3}\,\frac{\alpha E T_c}{\sigma_o(1-\mu)}\,\frac{a}{\rho} \quad \ldots\ldots\ldots\ldots (9)$$

Although Eq. 9 seems to be complex, the function which it represents can be used readily if properly plotted, because it is essentially composed of the following three factors:

Factor of structural loading: $X = \dfrac{p_o - p_i}{\sigma_o}$

Factor of plastic front radius: $Y = \dfrac{\rho}{a}$

Factor of temperature effect: $Z = \dfrac{\rho \, E \, T_c}{\sigma_o \, (1 - \mu)}$

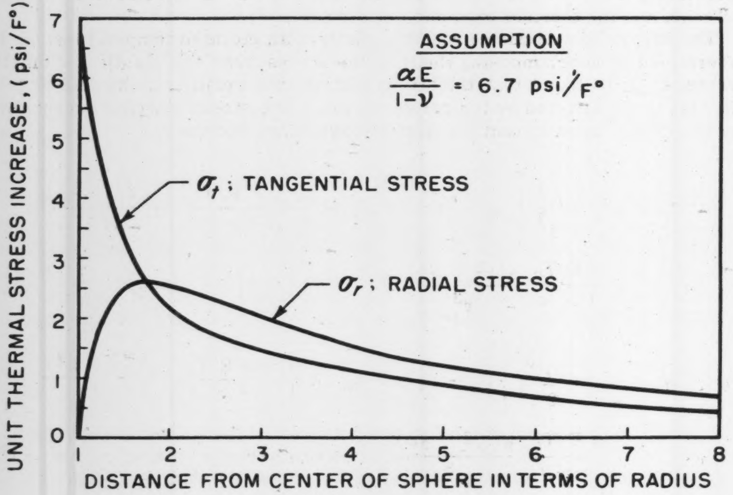

FIG. 5.—DISTRIBUTION OF UNIT THERMAL STRESS INCREASE AROUND SPHERICAL SALT CAVITY PER UNIT TEMPERATURE RISE IN CAVITY

Since the Z-value is constant at any given temperature, Eq. 9 can best be plotted on an X-Y diagram as shown in Fig. 6, using the following assumptions:

$$\frac{\alpha \, E}{1 - \mu} = 6.7 \text{ psi per } {}^\circ\text{F}$$

σ_o = 1,500 psi in temperature range up to 300° F
= 1,300 psi in temperature range from 300° F to 500° F
= 1,000 psi in temperature range from 500° F to 800° F
= 800 psi in temperature range from 800° F to 1,100° F
= 600 psi in temperature range from 1,100° F to 1,475° F

As seen in Fig. 6, a plastic zone will develop in a spherical cavity without any structural stress whenever cavity temperature is elevated over 800° F. Furthermore, the development of a plastic zone is much larger around a cylindrical cavity than a spherical one, with a rise in cavity temperature.

The amount of cavity volume reduction due to elevated temperature and the plastic flow of salt is calculated for both spherical and cylindrical cavities. Cavity volume is reduced by the thermal expansion of salt in the plastic zone, resulting in the plastic flow of the expanded salt into the cavity. The relation among the cavity temperature rise, the factor of plastic front radius, and cavity volume reduction is obtained by equating the salt expanded into the cavity with the cavity volume reduction.

In a spherical cavity:

$$\frac{\rho}{a} = \sqrt{\frac{2}{9\,\alpha\,T_c}\,(1-\zeta) + 1} \;\ldots\ldots\ldots\ldots\ldots \;(10)$$

in which ζ is the ratio of reduced cavity volume to the original volume, $1 = \zeta$ denotes the rate of cavity volume reduction, and α is the coefficient of the linear expansion of salt.

Under a steady heat flow from spherical cavities, a direct relationship between the factor of structural loading and the ultimate reduction of cavity volume, with any given temperature rise, can be obtained by eliminating the factor ρ/a from Eqs. 9 and 10.

In the case of a spherical cavity, the relation is found as:

$$\frac{p_o - p_i}{\sigma_o} = \ln\left[\frac{2(1-\zeta)}{9\,\alpha\,T_c} + 1\right] + \frac{2}{3} - \frac{2}{3}\,\frac{\alpha\,E\,T_c}{\sigma_o(1-\mu)}\sqrt{\frac{9\,\alpha\,T_c}{9\,\alpha\,T_c + 2\,(1-\zeta)}} \;\ldots\;(11)$$

In the case of a cylindrical cavity, the relation is found from an identical derivation as:

$$1 - \zeta = 3\,\alpha\,T_c\left(A\,e^{B} - 1.072\right)\ldots\ldots\ldots\ldots\;(12)$$

in which

$$A = 0.928 - \frac{B}{13.8}\;\ldots\ldots\ldots\ldots\ldots\;(13a)$$

$$B = \frac{2\left(\frac{p_o - p_i}{\sigma_o}\right) + 0.928\left[\frac{\alpha\,E\,T_c}{\sigma_o\,(1-\mu)}\right]\frac{2}{\sqrt{3}}}{\frac{2}{\sqrt{3}} + 0.072\left[\frac{\alpha\,E\,T_c}{\sigma_o\,(1-\mu)}\right]}\;\ldots\ldots\;(13b)$$

To determine an allowable temperature rise in a given cavity, Eqs. 11 and 12 can be utilized. The equations are plotted in Fig. 7, showing the relations of structural loading, maximum allowable temperature rise in a cavity, and reduction of cavity volume at equilibrium.

These analyses demonstrate the fact that a salt cavity will close completely when plastic flow occurs due to increased temperature. The closure will oc-

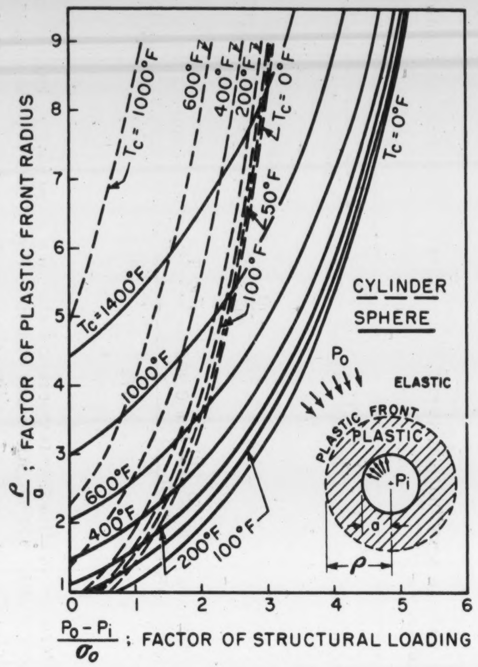

FIG. 6.—DEVELOPMENT OF PLASTIC ZONE AROUND SPHERICAL AND CYLINDRICAL CAVITIES WITH VARIOUS TEMPERATURE RISES

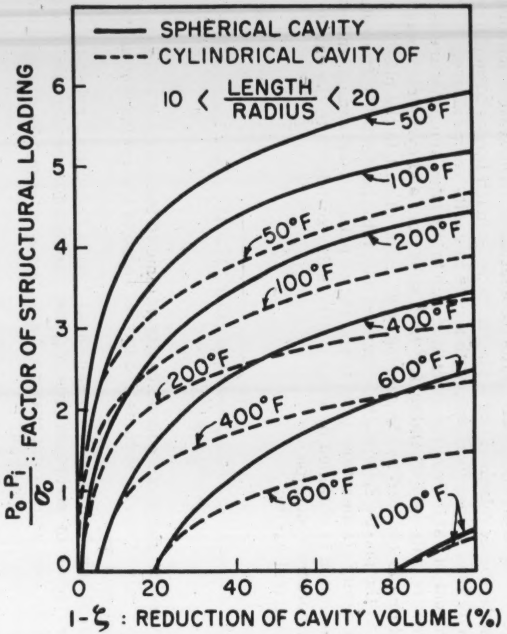

FIG. 7.—RELATION BETWEEN ULTIMATE REDUCTION OF CAVITY VOLUME AND STRUCTURAL LOADING WITH VARIOUS TEMPERATURE RISE IN CAVITY

cur before the cavity temperature reaches the melting point of salt (1,475° F.) Thus, cavity volume reduction due to elevated temperature can become a practical limitation. Therefore, the cavity can be successfully used for waste disposal if the cavity temperature rise can be controlled.

The ultimate structural equilibrium of an irregularly shaped cavity can be approximated by comparisons with the ideal spherical and cylindrical cavities. A cavity of irregular form would probably be as stable as a cavity of ideal form, since geometrical irregularities will not produce higher stress concentration because of the plastic nature of salt.

STRUCTURAL PROPERTY OF SALT

For application of the analytical theory of salt cavity equilibrium (Eqs. 11 and 12), various structural coefficients of salt have to be determined in the laboratory. In addition to the fundamental properties of salt, various factors affecting salt are investigated regarding reactor fuel waste storage in the salt cavity. The testing apparatus used is shown in Figs. 8, 9, 14, 17, 18, 20, and 21.

Consistent compression test results can be obtained only when the end friction effect of the salt specimen is eliminated. By using a specially developed standardized test procedure, the following fundamental properties of the aggregate salt from the Grand Saline salt mine were determined:

The maximum compressive stress	2,300 psi
with the standard deviation	200 psi
The 0.5% yielding stress	2,000 psi
Young's Modulus	0.14×10^6 psi
with the standard deviation	0.03×10^6 psi
Poisson's ratio	
with the compressive stress up to 300 psi	0.25 - 0.5
with the compressive stress over 300 psi	0.5

There is a wide variation in the previously reported figures regarding the fundamental properties of salt. These discrepancies are probably due to inconsistent testing procedures used by the various investigators.

The total creep of salt consists of instantaneous, transient, and steady state movements. It can be formulated with respect to time:

$$\epsilon_0 = \alpha + \beta \log t + \gamma t \ldots\ldots\ldots\ldots\ldots \tag{14}$$

in which ϵ_0 is the total creep, α_0 denotes the instantaneous creep, β is the coefficient of transient creep, and γ represents the coefficient of steady state creep.

Heat has the greatest effect upon the structural property of salt. The creep rate of salt is increased 75 times by a temperature increase of 80°F to 770°F. This means that a salt cavity reaches its ultimate structural equilibrium much faster if the cavity temperature is elevated. The equivalent yielding stress of salt is reduced to 1,000 psi with temperature increase of 80°F to 500°F; thereafter, the equivalent yielding stress remains nearly constant at around 1,000 psi as the temperature approaches the melting point of salt. Heat effects Young's modulus and the coefficient of linear expansion of salt very slightly. The specimens tested are shown in Figs. 15, 16 and 19.

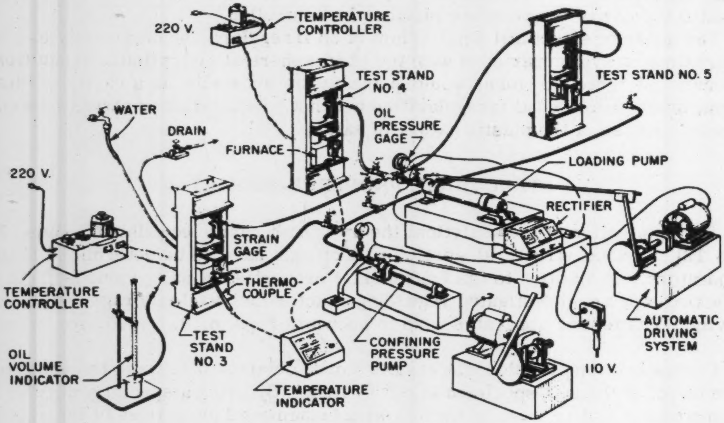
220 V.
TEMPERATURE CONTROLLER
TEST STAND NO. 4
TEST STAND NO. 5
WATER
DRAIN
OIL PRESSURE GAGE
FURNACE
LOADING PUMP
220 V.
RECTIFIER
STRAIN GAGE
THERMO-COUPLE
TEMPERATURE CONTROLLER
AUTOMATIC DRIVING SYSTEM
TEST STAND NO. 3
CONFINING PRESSURE PUMP
OIL VOLUME INDICATOR
110 V.
TEMPERATURE INDICATOR

FIG. 8.—ASSEMBLY OF COMPREHENSIVE TEST DEVICE

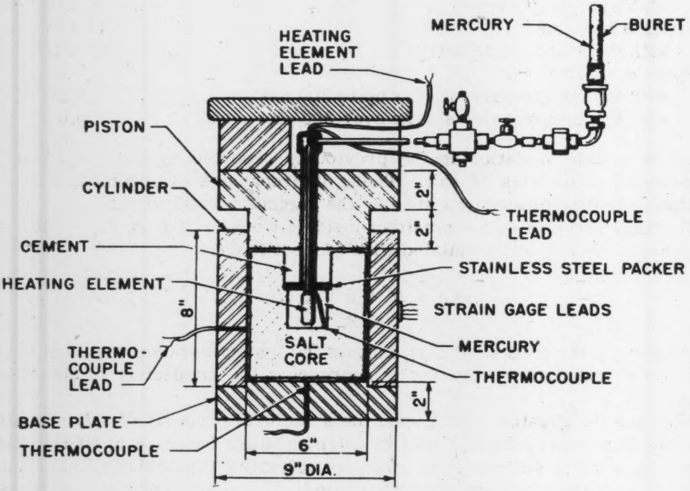
HEATING ELEMENT LEAD
MERCURY
BURET
PISTON
CYLINDER
2"
2"
THERMOCOUPLE LEAD
CEMENT
STAINLESS STEEL PACKER
HEATING ELEMENT
8"
STRAIN GAGE LEADS
SALT CORE
MERCURY
THERMO-COUPLE LEAD
THERMOCOUPLE
2"
BASE PLATE
THERMOCOUPLE
6"
9" DIA.

FIG. 9.—CROSS SECTION OF TEST CYLINDER

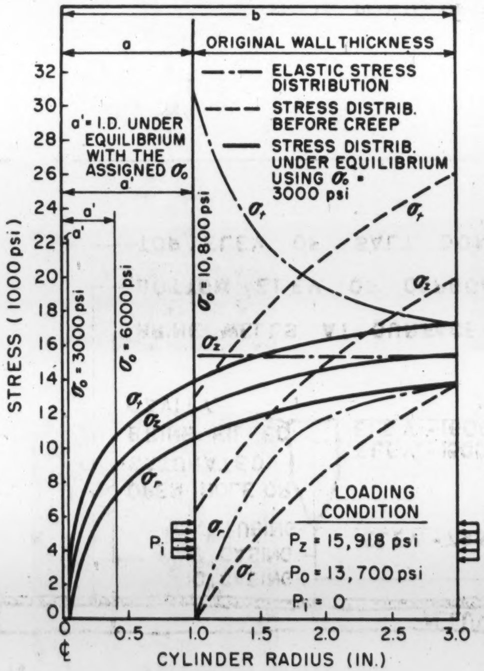

FIG. 10.—STRESS DISTRIBUTION AND REDUCTION OF RADIUS IN PROCESS OF CREEP CLOSURE OF CYLINDRICAL CAVITY

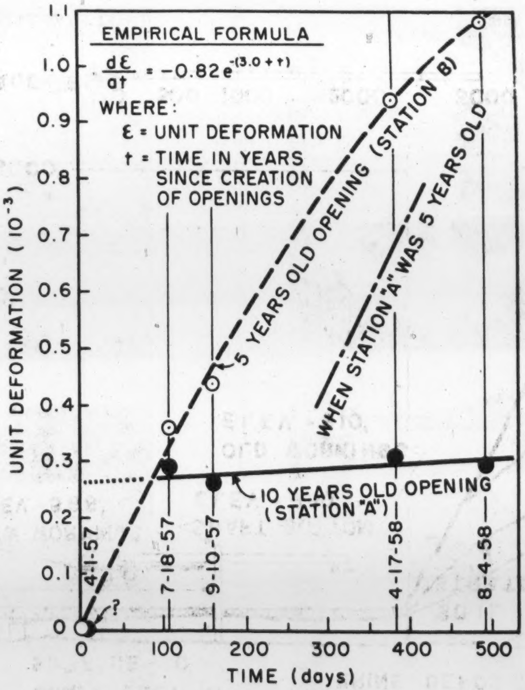

FIG. 11.—CREEP CLOSURE RATE OF TUNNEL OPENINGS IN GRAND SALINE SALT DOME AT DEPTH OF 700 FEET

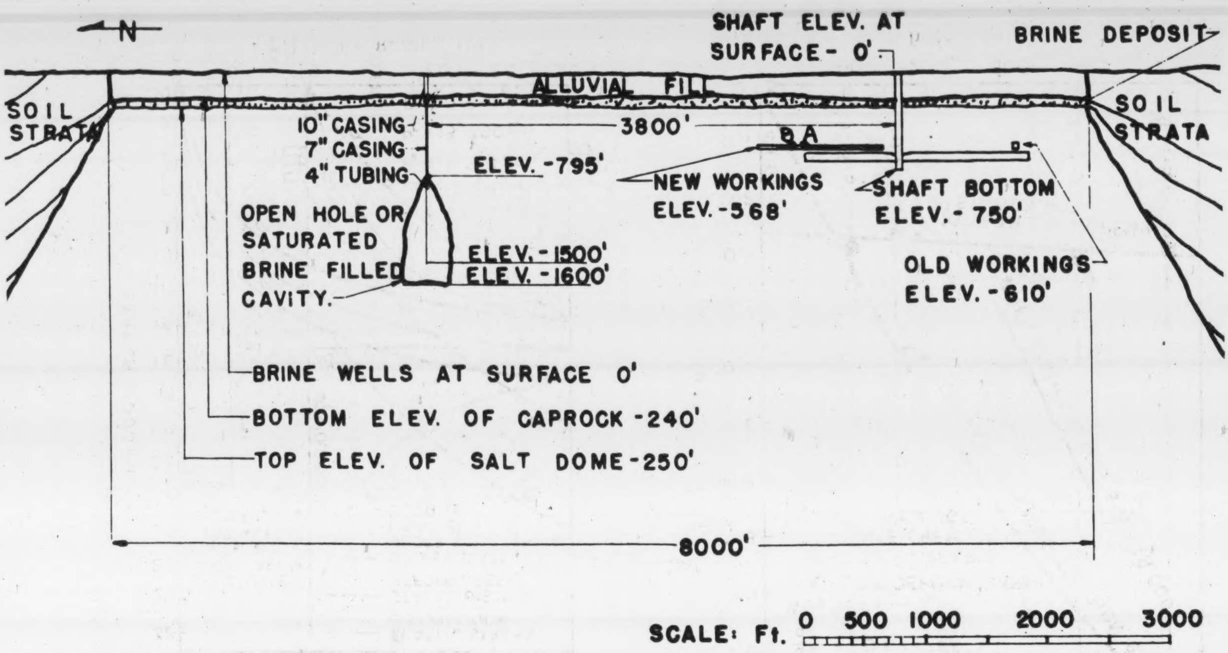
N
SHAFT ELEV. AT SURFACE - 0'
BRINE DEPOSIT
ALLUVIAL FILL
SOIL STRATA
SOIL STRATA
10" CASING
7" CASING
4" TUBING
3800'
D.A.
ELEV. -795'
NEW WORKINGS ELEV. -568'
SHAFT BOTTOM ELEV. - 750'
OPEN HOLE OR SATURATED BRINE FILLED CAVITY.
ELEV. -1500'
ELEV. -1600'
OLD WORKINGS ELEV. -610'
BRINE WELLS AT SURFACE 0'
BOTTOM ELEV. OF CAPROCK -240'
TOP ELEV. OF SALT DOME -250'
8000'
SCALE: Ft.
0
500
1000
2000
3000

FIG. 12.—PROFILE OF GRAND SALINE SALT DOME

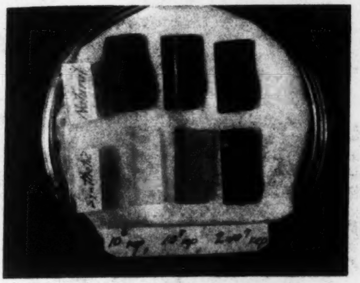

FIG. 13.—DARKENING EFFECT OF GAMMA IRRADIATION UPON NATURAL AND SYNTHETIC SALT CRYSTAL

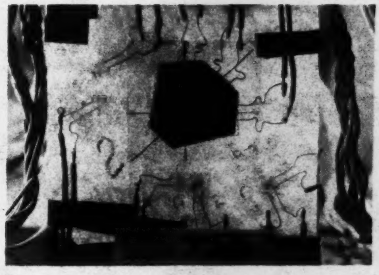

FIG. 14.—SR-4 STRAIN GAGES; TYPE AR-1 FOR STRAIN IN AGGREGATE AND TYPE A-8 FOR STRAIN IN GRAINS

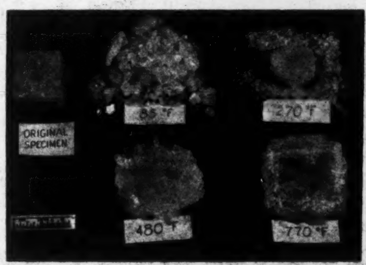

FIG. 15.—SPECIMEN TESTED BY UNIAXIAL COMPRESSION SHOWING INCREASE OF DUCTILITY WITH INCREASE OF TEMPERATURE

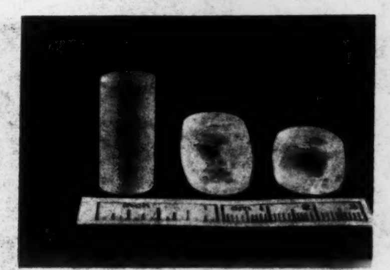

FIG. 16.—CRYSTAL FROM GRAND SALINE SALT MINE TESTED WITH CONFINING PRESSURE OF 14,500 psi AT 300°F AND 570°F. (LEFT, ORIGINAL; MIDDLE, 300°F, RIGHT, 570°F)

FIG. 17.—CREEP TEST AND HEATING APPARATUS

FIG. 18.—CREEP TEST OF STANDARD SPECIMEN SUBMERGED IN REACTOR FUEL WASTE SUBSTITUTE. (ALUMINUM NITRATE SOLUTION)

FIG. 19.—PLASTIC DEFORMATION OF SPECIMEN BY UNIAXIAL COMPRESSION AT 770°F

FIG. 20.—400,000 POUND TEST APPARATUS FOR MEASURING CREEP RATES IN CYLINDRICAL CAVITY

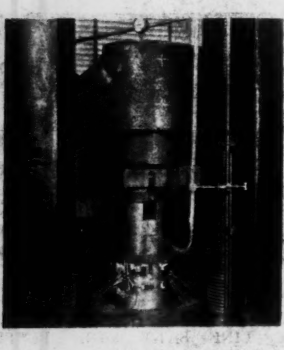

FIG. 21.—CYLINDRICAL CAVITY TEST APPARATUS

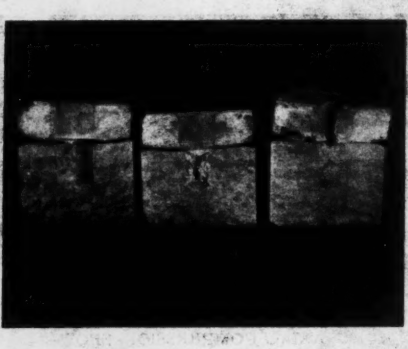

FIG. 22.—CLOSURES RESULTING FROM CREEP IN CYLINDRICAL SALT CAVITY. (6" O.D x 2" I.D)

FIG. 23.—PARTIALLY CLOSED CYLINDRICAL CAVITY

FIG. 24.—THE 98% CAVITY CLOSURE

Gamma irradiation of salt up to 20 x 10^6 rep has no effect upon the fundamental structural properties of salt. However, irradiated salt exhibits a slight reduction in the rate of transient creep. Examples of irradiated salt crystals are shown in Fig. 13.

The maximum stress of salt is not affected by its submersion for one month periods in light mineral oil, reactor fuel waste substitute (nitrate waste), and saturated salt water. Salt submerged in the saturated salt water exhibited some increase in creep, although no similar effect was found in the salt submerged in the reactor fuel waste substitute. A submerged specimen is shown in Fig. 18.

LABORATORY EXPERIMENT OF CAVITY CLOSURE

The analytical theory of cavity equilibrium was tested in the laboratory by using a model salt cavity. A cylindrical hole 2 in. inside diameter and 2 in. deep was cored out of the center of a cylindrical salt specimen of 6 in. outside diameter and 6 in. high. As seen in Fig. 9, this specimen was fitted tightly into a stainless steel cylinder. A triaxial stress condition is created around the cylindrical salt cavity simply by giving axial pressure, since the lateral pressure is produced by reaction of the steel cylinder against the lateral expansion of the salt. This lateral pressure was measured by strain gages attached to the outside of the steel cylinder, as shown in Figs. 9, 20, and 21.

About 16,000 psi of axial compressive stress was produced around the salt cavity by applying a vertical load of 400,000 lb on top of the specimen. The rate of cavity closure was determined by measuring the displacement of mercury from the cavity while the vertical load was kept constant. This loading was continued until the rate of the closure approached zero as a result of the creep equilibrium established around the cavity.

Analysis of the cavity closure and stress distribution is presented in Fig. 10. Assuming that the salt behaves as an elastic under this compression, the maximum differential stress was calculated to be 31,000 psi. Since such a large differential stress cannot exist, a stress-distribution calculation was made assuming that the salt deformed plastically. As shown in Fig. 10, the difference between the plastic stress distribution and the elastic stress distribution is noticeable.

By assuming plastic deformation, the maximum differential stress in the salt is reduced from 31,000 psi to 13,000 psi, which is obviously still much higher than salt can withstand. This excess differential stress becomes the driving force for closure of the cavity by creep. The equilibrium condition is established by reducing the cavity volume, resulting in an increase in the factor describing the plastic front ρ/a. This process of establishing a structural equilibrium is, in principle, the same as that for the development of a plastic zone. The process of the cavity closure is shown in Figs. 22, 23, and 24.

ACTUAL CREEP MEASUREMENT IN THE GRAND SALINE SALT MINE

In order to confirm the previously developed theoretical conclusion about the structural equilibrium of a salt cavity, the actual creep in the Grand Saline salt mine was measured at a depth of 700 ft. The self-explanatory pictures of the mine are shown in Figs. 25 to 30. A device, as shown in Fig. 25, equipped with a dial gage, was designed to measure the rate of closure. Two locations in the mine, A and B, which were opened at different times (A in 1947, and B in 1952) were chosen for measurement of the closure rate, as shown in Fig. 12. By selecting these two stations, the effect of age upon the closure rate was studied. The SR-4 strain gages used for creep measurement failed because of gage corrosion. The test device is shown in Fig. 27.

As the result of measuring creep for over a year, a remarkable difference in the creep rate of the new and old opening was found, as seen in Fig. 11. This difference in the creep rate indicates a reduction of creep rate with time. From the experimental data, the following empirical formula for the creep rate is found:

$$\frac{d\epsilon}{dt} = -\ 0.82\ e^{-\ (t + 3.0)} \quad \ldots\ldots\ldots\ldots\ldots \quad (15)$$

This equation indicates that the creep in the mine will soon be extinguished.

The total amount of strain in the opening can be calculated from integration of the above creep rate equation as:

$$\epsilon = \int_{t_1}^{t_2} d\epsilon = -8.2 \int_{t_1}^{t_2} e^{-\ (3.0 + t)}\ dt \quad \ldots\ldots\ldots \quad (16)$$

The ultimate total strain can be obtained by taking $t_1 = 0$ and $t_2 = \infty$ as:

$$\epsilon = \int_{0}^{\infty} d\epsilon = 0.82\ e - 3 = -\ 4.1\%$$

Timber beams broken by the creep are often found in the older openings of the mine as shown in Fig. 29. Measurement of the broken beams indicates that the openings have shrunk 1% to 2% over the past 15 yr to 20 yr. This agrees with the above calculation if the beams were erected there several years after the mine was opened.

The extinction of the mine creep and the nature of the empirical creep equation are positive proof of the fact that the mine opening establishes a structural equilibrium by developing plastic zones around the corners of the opening where high stress concentrations develop. All the evidence observed in the mine not only agrees with the theoretical conclusion of structural cavity equilibrium but also points favorably to stability of the cavity for waste storage use.

DESIGN PRINCIPLE

The principle for designing a salt cavity can be derived from the previously described theoretical and experimental investigations. The structural equilibrium of a salt cavity can be evaluated from these analyses, assuming that a chemical equilibrium between the salt and the waste does not become paramount.

The structural equilibrium of a salt cavity can be determined from three principal factors: the structural loading factor, cavity volume reduction, and the temperature rise in the cavity. Any one of these three factors can be determined from Fig. 7 if the other two are known.

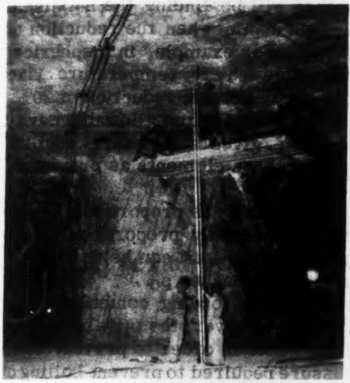

FIG. 25.—STATION B; MEASURING CREEP CLOSURE RATE OF MINE OPENING BY USING DIAL GAGE DEVICE

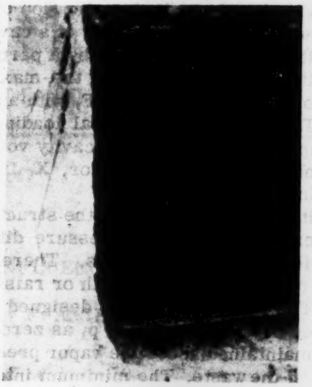

FIG. 26.—TYPICAL SUPERFICIAL SCALING WITH SHARP ANGLE OF FRACTURE DEVELOPED AT VERTICAL CORNER OF PILLAR

FIG. 27.—POINT NO. 1; SR-4 STRAIN GAGE MEASUREMENT ON WALL OF MINE

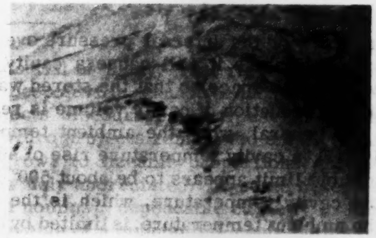

FIG. 28.—IMPURITY STRATA INDICATING PREVIOUS HISTORY OF DEFORMATION

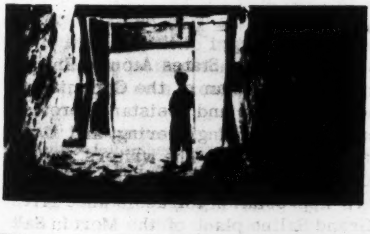

FIG. 29.—BROKEN TIMER FRAMES RESULTING FROM CREEP

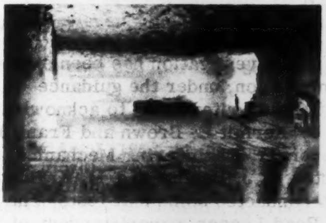

FIG. 30.—MINE OPERATION AT 700 FT. BELOW SURFACE IN GRAND SALINE SALT DOME IN EAST TEXAS

The most practical application of this method will be finding the maximum allowable temperature rise of a cavity at a given depth, when the reduction of cavity volume is limited by a particular value. For example, in a spherical cavity at depth of 1,000 ft, the maximum allowable cavity temperature rise is found to be around 500°F, with an expected cavity volume reduction of 20%.

The factor of structural loading is directly related to the allowable cavity temperature rise and the cavity volume reduction, given a structural equilibrium condition. This factor, X, is composed of three elements as previously defined.

It should be noted that the structural load on a cavity is proportional to the external and internal pressure difference, and inversely proportional to the equivalent yielding stress. Therefore, the structural load can be reduced by shortening the cavity depth or raising p_i until it approaches p_o.

The cavity should be designed for the maximum loading condition at any given depth by assuming p_i as zero. In practice, however, the p_i value should be maintained above the vapor pressure of the waste in order to prevent boiling of the waste. The minimum internal pressure required to prevent boiling of the waste is a little less than the vapor pressure of water, which is as follows:

Temperature	The Vapor Pressure of Water
212° F	15 psi
300° F	58 psi
400° F	250 psi
500° F	681 psi
600° F	1,553 psi
700° F	3,100 psi

Increase of the internal pressure over the external pressure must not be allowed, because it will cause a cavity fracture. A cavity should be designed with excess space so that the stored waste will not be forced out when the ultimate reduction of cavity volume is reached.

In general, when the ambient temperature of the salt formation is around 100° F, a cavity temperature rise of 400°F is permissible. The maximum allowable limit appears to be about 500° F in such a cavity. On the other hand, the cavity temperature, which is the sum of the cavity temperature rise and the ambient temperature, is limited by the vapor pressure of the waste. Therefore, the cavity temperature should be maintained below the point where the vapor pressure of the waste is equal to the overburden pressure.

There is no structural restriction upon the size of a cavity, except that the cavity radius should not exceed one-third of the overburden depth. Therefore, in most cases, the size of the cavity should be determined in relation to the cavity temperature rise.

ACKNOWLEDGMENTS

This investigation has been sponsored by the United States Atomic Energy Commission, under the guidance of Joseph A. Lieberman of the Commission staff. The writers wish to acknowledge the cooperation and assistance provided by Kermit E. Brown and Frank Jessen, Petroleum Engineering, and E. A. Ripperger, Engineering Mechanics, University of Texas. The writers are indebted to J. Handin and M. K. Hubbert of the Shell Development Laboratory who conducted some salt tests. The writers' appreciation for assistance given by Read Lesser, superintendent of the Grand Saline plant of the Morton Salt Company, in supplying salt specimens and furnishing men and machines for experiments conducted at the mine is acknowledged.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3147

HYDROELECTRIC POSSIBILITIES AND INFLUENCE OF LOAD GROWTH

By S. P. McCasland,[1] F. ASCE

SYNOPSIS

This paper is devoted to the physical and economic enhancement that is imparted to hydroelectric potentialities by expansion of the power markets to which they are accessible. Particular emphasis is given to the improvement which small undeveloped sites may derive from the rapid increase in demand which is being encountered by many young power systems in foreign countries.

It is widely known that the power potential of many great rivers, such as that of the Congo with 25 million kw, at Leopoldville, cannot be harnessed because of the minimum size of the projects discourages their use their the present. The fact that many potential hydro projects remain too small to be attractive until the accessible market grows to sizeable proportions, on the basis of steam-generated power, is not widely appreciated. In some cases, a site which might have contributed only a nominal, dependable capacity to the load of the early 1920's, may become capable of a substantial contribution to the present-day load. As explained in detail subsequently, the larger load offers a shorter peaking period for minimum-stream flow and, thus, increases potential firm capacity; a bigger load can also absorb more hydro energy at times of high-stream flow. Outstanding enhancement is not, by any means, imparted in all cases of load growth. Occasionally, however, a site which was quite properly rejected long ago, as too small to be of interest, may be found to have attained very substantial capability. However, such increased hydro capability is of little importance to the power consumer, unless it makes possible the production and sale of hydro power at a rate cheaper than some other source, such as a steam boiler.

Note.—Published essentially as printed here, in April, 1960, in the Journal of the Power Division, as Proceedings Paper 2435. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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Stream flow and available head have value only in relation to the power system into which the hydro output can be incorporated, and the demand which it can serve. For any given load, a particular site—with definite head and water supply—is potentially capable of a certain contribution in both capacity and energy. With any other load, however, this potential capability would be different; in other words, capability is a function of load as well as of water and head. If a plant is not constructed with enough generating capacity, the potential capability may not be fully realized; but regardless of the magnitude of hydro generators, no more than the potential capability can be attained for the same combination of load, water, and head. No engineer would design a bridge without knowing the load, and this knowledge is just as essential to a hydro analysis, even though the term "load" means something quite different in the latter case.

In considering prospects for the development of available head and water for a particular load, the engineer is evidently faced with the necessity of comparing hydro cost with that for the cheapest alternative means of power production. In practice, this usually means a comparison of hydro cost with that for steam or diesel-generated power. In the areas under discussion there is usually no opportunity to incorporate auxiliary benefits, such as flood control, and the hydro power must stand on its own merits. As inferred previously, there is usually no reason to undertake a hydro development if it does not have a cost advantage. A hydro project ordinarily entails a heavy initial outlay for civil engineering works because the dam, tunnels, etc., must be built to ultimate size at the outset, even though the installation of some of the generating units may be postponed. Also a transmission line must be built from the hydro site to the load center. Although the incremental cost of adding future units may be small, and the project's final average cost per kilowatt may be correspondingly low, interest on the early investment, during the years before the complete installation is needed, may be a critical factor. For example, it might be decided that X River could be developed to meet load growth from 1965 to 1980, by the progressive installation of three 10,000 kw units in 1965, 1970, and 1975. It might be estimated that the final investment in the ultimate project would be $6,000,000, which might seem to be quite a bargain, being only $200 per kw. But it might cost $5,000,000 to get the first unit on the line, even though the other two would come along for $500,000 apiece when they would be needed. Thus, the project would be saddled with interest on some part of $5,000,000 for 15 yr before the investment would be fully productive. With money at 10% or even 12%, which is not unusual for foreign installations, interest could be quite a factor in actual cost. Thermal additions, on the other hand, have a more uniform cost per kilowatt, and consequently entail a more gradual rate of investment with a lesser penalty in interest charges. Particularly for small power supplies, the initial hydro expenses may be prohibitive and for many young systems, thermal generation has been clearly the best course from the standpoint of both the utility and its customers. With expansion of the system by such thermal additions, however, a small and formerly unattractive hydro site may have become sufficiently enhanced to be competitive as the next step in extension of power production facilities. For example, suppose the development of X River had been postponed for 30 yr. The capability of the site might be increased to 50,000 kw under a load sufficiently large to call for the full 50,000 kw in a single step at the outset. Obviously the hydro plant would be more attractive under the latter conditions.

There are factors outside the fields of economics and engineering which influence the development of power systems, and these are sometimes accentuated in foreign countries. An example of the peculiar situations which can develop may be of interest. In some parts of the world, the family is still about the largest unit of private business because social custom does not afford a ready opportunity for the formation of local corporations or even partnerships on a large scale. In addition, there are factors which frequently discourage the entrance of interests from outside the country. In these circumstances, one prominent middle eastern city of about 1,000,000 people came to be served, in the mid 1950's, by more than twenty-five independent power systems, each covering a segment of the city. There had been no entity with enough resources to cope with the load as a whole, and most of the power systems simply represented the limit of family finances. In most instances power was scarce and had to be rationed, even though it was customary for the rate per kilowatt-hour to ascend with greater consumption, instead of descending in the usual manner. There, and in some other parts of the world, power has been regarded very largely as a means of providing light. A sizeable block of customers on such systems still may consume only about 10 kwh a month. Plants are apt to be shut down during part or all of the daylight hours, and rates up to $0.20 per kwh, or even more, are not unknown.

While demand is still subject to these "growing pains" in some parts of the world, the situation has changed rapidly since World War II. In most foreign areas where service has been good and rates reasonable, the post-war increase in demand has been phenomenal. There has been an increased interest in appliances such as stoves, irons, and air conditioners, and there has also been a pronounced acceleration in the trend of industrialization. Industrialization provides factory employment, servants become harder to get, and appliances become more popular. In some areas of the Americas, compound annual-load growth of 12% or even 14% has occurred. At such rates, even a small market can become a big one in an astonishingly short time, and sometimes formerly discarded hydro sites can attain importance, as a result.

In considering the power which can be derived from a hydro site, it is all-important to differentiate sharply between potential capacity and potential energy; that is, between kilowatts and kilowatt hours. With the vast amount of interconnection, diversity, and reserve that exists in the United States, it is often the custom to wholesale power on the basis of energy alone, and there is sometimes a tendency to lose sight of capacity value. But the utility that distributes and retails the power can never be indifferent to this vital factor, and the wholesale price always reflects it. Particularly in a small isolated system overseas, where a single hydro plant can be a substantial component, its contribution to capacity is a matter of physical necessity. If the hydro plant does not measure up to expectations on some dry day, a part of the load has to be rejected with all the undesirable consequences of injured public relations, criticism in the press, and perhaps even real damage to some customers. A shortage of energy, on the other hand, simply means that some steam plant has to burn more fuel for a time, with a strictly temporary debit on the books. Further comment as to energy production is offered subsequently, but first consideration will be given to capacity, which is by far the most important component of demand in the areas under discussion. The capacity which any particular hydro site can contribute to a given load with reliability is known as "firm" capacity. It is governed by the water supply that will be available in the greatest drought reasonably expected, that is, the driest day of the driest year, or if there is storage, the extent of the driest period likely to occur. An estimate of firm capacity thus calls for a determination of minimum water supply, and this is one of the most troublesome problems in foreign hydro work. Although there are outstanding exceptions, satisfactory records of stream flow are not usually available. In some cases, existing records even have been thrown away by newly appointed officials who did not appreciate their value. When a private or public entity becomes especially interested in a particular site, it is, of course, usual to gage the water. But such keen interest is apt to develop only when the site comes under active consideration, and not decades earlier. The idea seems to prevail, even among engineers in other fields, that hydro engineers have some sort of crystal ball for determination of water supply. In the United States there might be a 50-yr record, and the sponsor of a project would have, as a calculated risk, at least an even chance that no worse conditions than those recorded would occur during the economic life of the plant. But if there had to be 50 yr of records, there would be very few hydro plants in the areas under discussion. One has to use whatever data he can get. Sometimes the projects are under-built because of a too-conservative estimate.

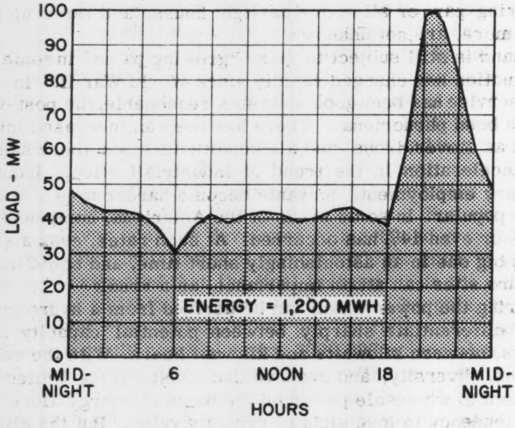

FIG. 1.—TYPICAL DAILY LOAD CURVE, 50% LOAD FACTOR

Conversely, they may be over-built. Sometimes the operators can fall back on reserves in time of drought. And sometimes there is a power shortage. After the installation of a hydro plant, however, further load growth is often met by thermal additions; this usually improves the hydro capability and the significance of water shortage may diminish considerably. An unexpectedly critical drought may not occur for many years after the project goes into service, and during that time the hydro capability may be strengthened enough to contribute its full installed capacity, even with a lesser minimum water supply than anticipated.

The basis for improvement in potential power capability is illustrated graphically by Figs. 1 through 5. Fig. 1 shows a typical daily load curve for a small system. The ordinate of the curve represents the demand on capacity, and the abscissa, time; the area obviously is energy. The relation of energy to capacity is expressed by the "load factor," which may be defined as the ratio of average load to maximum load; this factor is considerably lower in young systems than in the United States. In the case illustrated, the load factor is about 50%. At 6 A.M. demand is at the minimum for the day, 30 mw; this is called base load. At about 8 P.M. the peak demand of 100 mw occurs.

For illustration, suppose the load is met by four identical 25 mw steam generators as in Fig. 2. On the day illustrated, the four generators would have to produce a total of 1,200 mwh of energy. One, labelled "D", would operate continuously on the base of the load and would have to generate half this amount, namely 600 mwh. However, generator "A," operating on the peak, would be in service for only about 3 hr and it would be operating at full capacity for only a small part of that time. It would be called on to produce only 48 mwh. Although this generator constitutes 25% of the required capacity, it must produce only 4.0% of the required daily energy, and its boiler will consume less than 10% of the fuel needed in the case of generator "D." Now let us move 5 yr into the future, and assume that the load of each hour has doubled, with two 50 mw units added to the system. The increased load is shown by Fig. 3. Generator "A" now is only $12\frac{1}{2}\%$ of the total capacity and it need produce only 32 mwh. This is two-thirds as much energy as the same generator was required to produce 5 yr before, and only 1.3% of the daily energy in the larger load. This relation between percentage of capacity and corresponding energy, expressed by what may be called a "peak percentage" curve, is shown by Fig. 4. The scales are expressed in percentage and the curve applies to the loads of both Figs. 2 and 3; that is, to any magnitude of load having the same hourly variation. For a load which increases from year to year, a fixed number of kilowatts on the peak is obviously a smaller and smaller percentage of total operating capacity. The essence of the matter, however, is that this diminishing percentage of capacity is called on to produce a disproportionately smaller percentage of daily energy. In other words, the required percentage of energy production decreases at a

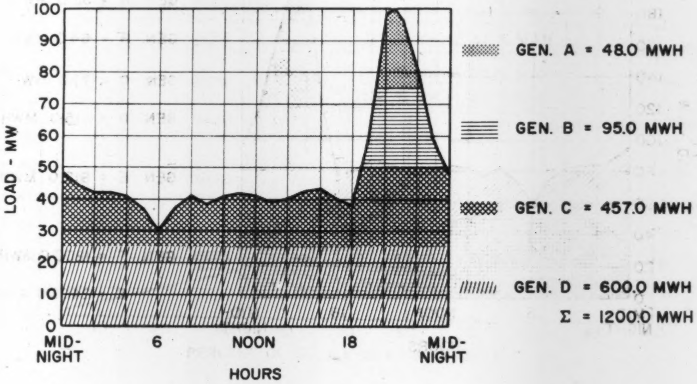

FIG. 2.—ALLOCATION OF PLANTS, 100 MW DAILY LOAD, 50% LOAD FACTOR

much faster rate than the percentage of capacity. Note that 25% of the peak capacity produces 4% of the daily energy; $12\frac{1}{2}\%$, 1.3% of the energy; and 5% only 0.3% of the energy.

Suppose Generator "A" had been a hydro unit. With the head available, the water supply on the minimum day represents a definite amount of daily energy. The stream flow can be accumulated in a small pond during off-peak hours and the total daily water supply can be released through the plant at will during a short peaking period. Generator "A" would have to produce 48 mwh in order to meet 25 mw of the 100 mw load as shown by Fig. 2. However, with the load increased to 200 mw as shown by Fig. 3, Generator "A" would be only $12\frac{1}{2}\%$ of the total capacity and it would be required to turn out only 32 mwh to meet 25 mw of the increased load; 32 mwh obviously can be produced with only two-thirds of the water required for the production of 48 mwh. Thus, doubling of the system load makes it possible for the same plant to maintain its full capacity with two-thirds of the water required in the case of the smaller load.

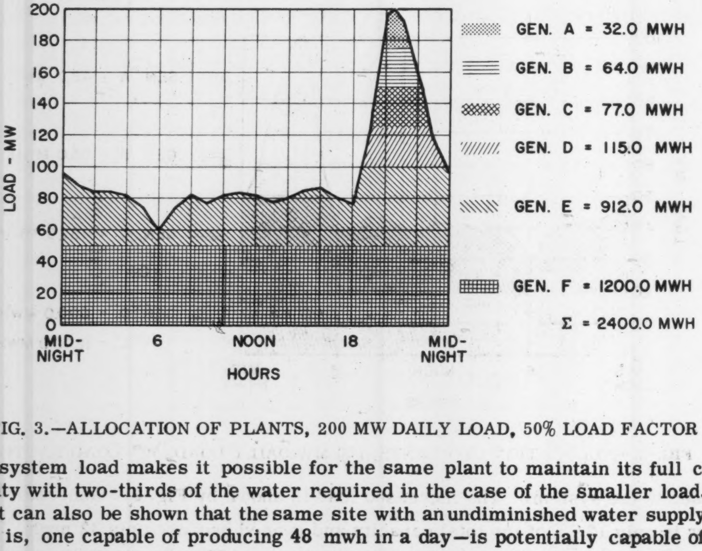

FIG. 3.—ALLOCATION OF PLANTS, 200 MW DAILY LOAD, 50% LOAD FACTOR

It can also be shown that the same site with an undiminished water supply—that is, one capable of producing 48 mwh in a day—is potentially capable of a greater contribution than 25 mw to the larger load. As shown by Fig. 4, 48 mwh is 4.0% of the daily energy in the 100 mw load, and, obviously, it is half that percentage, or 2.0% of the energy in the 200 mw load. From Fig. 4, it can be seen that the latter energy percentage will sustain 17.0% of the required peak capacity, or 17.0% of 200 mw. Thus, doubling the load increases the potential dependable hydro output from 25 mw to 34 mw with the same water supply, and the same site.

Fig. 5 illustrates an extreme case of how hydro capacity can increase with the same site and water supply. This hypothetical site offers 100 m of head and a water supply averaging 1.0 m^3/s, or about 21 mwh on the minimum day.

At the top of Fig. 5 there is a curve which illustrates the utilization of this energy at 60% load factor for an isolated load, where the one hydro plant would be the only source of power. Under such conditions the plant would have to be in operation for 24 hr a day and the maximum demand that the hydro plant could meet would be only 1.4 mw. It might not be worth while to build the hydro plant for such a small output. However, if steam additions were made to support load growth to the extent shown by the lower curve, the situation would be different. Here the hatched area "II" at the base of the curve represents the thermal energy produced by 40 mw of thermal capacity in extreme drought for a load of 50 mw having the same characteristics as above. With this thermal support, the hydro plant would be in operation for only about 3 hr, and the same minimum day hydro energy—21 mwh—could meet 10 mw of the load as shown by the hatched area "I" on the peak of the curve. In other words, this latter area has exactly the same size as that under the upper curve, but due to the shorter operating time with the larger load, the attainable hydro capacity is seven times the hydro capacity attainable in the situation shown by the upper curve. The hydro development might very well be justified on the basis of this larger output.

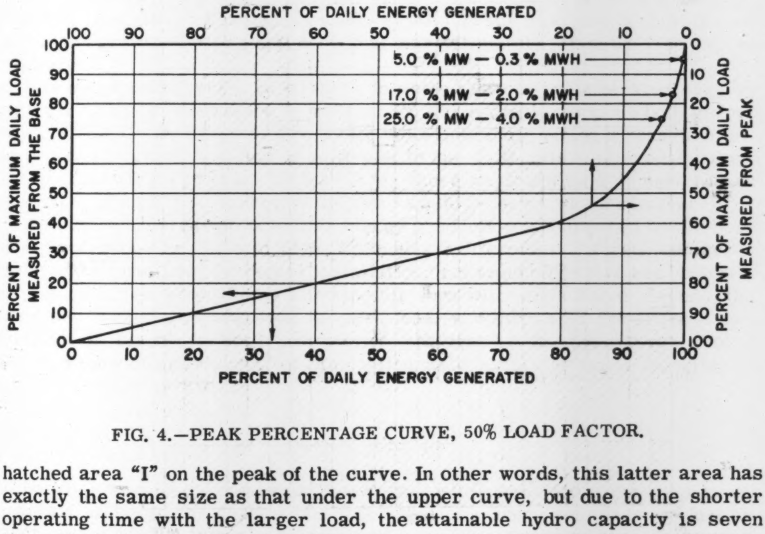

FIG. 4.—PEAK PERCENTAGE CURVE, 50% LOAD FACTOR.

Improvement in hydro capability, as described above, is more pronounced with low load factors and sharp peaks. It sometimes happens that load factors increase as the magnitude of the peak load increases, and also the peak may become more blunt. These conditions can diminish, and in extreme cases even entirely erase the gain in hydro capability. In most cases, however, the shape of the load curve tends to change slowly in relation to load growth. This is particularly true in young systems.

Up to this point, comment has related to the contribution of the hydro plant to system load-carrying capacity under the worst water conditions. In years

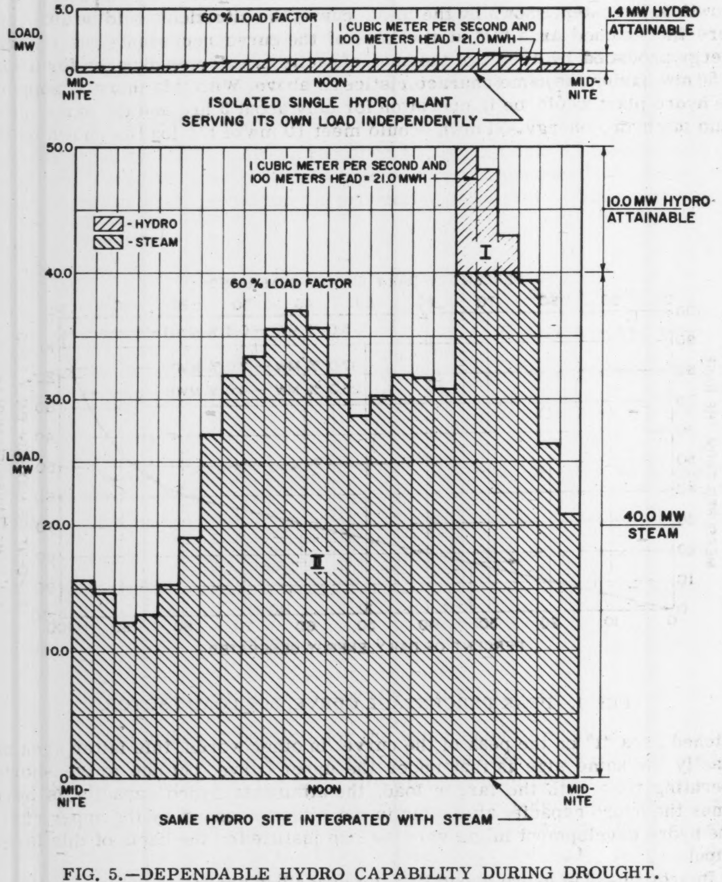

FIG. 5.—DEPENDABLE HYDRO CAPABILITY DURING DROUGHT. ISOLATED VS. STEAM-SUPPORTED. SAME HEAD-SAME WATER. DAILY LOAD CURVES AT 60% LOAD FACTOR

of critical drought, a hydro plant may produce very little energy and in a combined hydro-thermal system most of the energy component of demand probably will be produced with fuel. However, years of great drought are rare. In wet years a hydro plant may operate continuously in the position shown for unit "D" in Fig. 2, and produce half of the system energy. At such times thermal units are switched to the peak, where they contribute their full capacity with little fuel, just as hydro units on peak require little water. In normal years, hydro might occupy some intermediate position between the base and the peak as, for example, that for unit "B" or "C" in Fig. 2. Over the life of the hydro project, the quantity of hydro energy which can be produced is a function of the average water supply, and not the minimum which governs capacity.

A hydro plant's energy falls short of full use of the average water supply under two conditions. The first of these conditions exists when the uncontrollable stream flow exceeds the maximum volume that can be passed through the turbines, and spills past the plant intake. To convert such spill to useable energy, additional plant capacity may be installed. However, this additional capacity is inoperable in dry years and hence cannot help to meet the load. In some cases, such excess capacity can produce enough energy, on the average, to justify the cost of its installation only by the fuel saving at the thermal plants. Also load growth and thermal additions may ultimately "firm up" the excess capacity.

The second occurrence of waste is in water that can produce hydro energy only during hours when there is no demand. For example, suppose both generators "C" and "D," Fig. 2, are hydro units operating during a season of uncontrollable stream flow that is sufficient to run both units continuously. During the hours from midnight to 6:30 P.M., there would not be a demand for all the output of unit "C", consequently there would be some waste of water.

As the load increases, potential energy production is improved by the possibility of reducing waste in both of the above categories. First, the larger load makes practicable a bigger installation, consequently there are, potentially at least, fewer occasions when the turbines will reject water. Second, the base load increases along with the peak load, and this eases the off-peak restriction on the marketability of energy. Notice that for the 200 mw load of Fig. 3, 50 mw of hydro could be run continuously, while this was impossible in the 100 mw load of Fig. 2.

CONCLUSIONS

Hydro power must be competitive in cost with power from other sources, to be attractive to the consumer. The value of a hydro site depends just as much on the system into which its output can be incorporated and the demand it can serve, as it does on water supply and fall. A measure of the project's worth is, thus, the steam or diesel expense which can be avoided by its construction. The firm hydro capacity which can contribute with reliability to meeting the load makes it unnecessary to build equivalent thermal capacity, and the marketable hydro energy which can be produced renders unnecessary the fuel cost and other operating expenses incident to producing the same amount of energy by steam. If the present worth of estimated annual charges on the hydro plant—that is, the capitalized value of interest, amortization, taxes, insurance, and operation—is less than the present worth of similar charges and fuel expenses corresponding to the thermal power avoided, the hydro is the best buy. And even if the hydro is uneconomical today, it may become attractive at some future time.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3151

COOLING WATER FOR STEAM ELECTRIC STATIONS ON TIDEWATER

By R. W. Spencer,[1] F. ASCE, and John Bruce[2]

SYNOPSIS

The electric utility industry is familiar with the rapidly increasing size of steam electric stations and the enormous quantities of cooling water that must be moved to serve them. Designers recognize the importance of the many factors which must be considered in providing a system of lowest initial cost, and minimum operating expense, with a high degree of reliability and availability. This paper presents a brief review of a number of tidewater installations in California rather than an analysis of the detail of mechanical and hydraulic features of cooling water systems.

SOUTHERN CALIFORNIA EDISON COMPANY SYSTEM

As will be noted from the map (Fig. 1) all steam stations on the Southern California Edison Company system, with one exception, are dependent directly on the Pacific Ocean for cooling water. Of necessity several are situated at, or quite near to, resort areas and public beaches. State agencies and good public relations require the areas to be protected for propagation of fish, for ocean sports, for boating, and for general aesthetic enjoyment.

At four plants this has been accomplished by the installation of large concrete conduits extending well out into the open ocean and resting in excavated

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Note.—Published essentially as printed here, in June, 1960, in the Journal of the Power Division, as Proceedings Paper 2503. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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trenches in the sea bottom, although the primary reason for such location has been to avoid the hazard of damage to exposed structure by heavy seas. No provision is made for unwatering or direct cleaning of the conduits but antifouling measures are taken. Ten years' experience with this scheme has proved it to be reliable and satisfactory although somewhat more costly than cooling water arrangements in protected bays and rivers, when such arrangements are available.

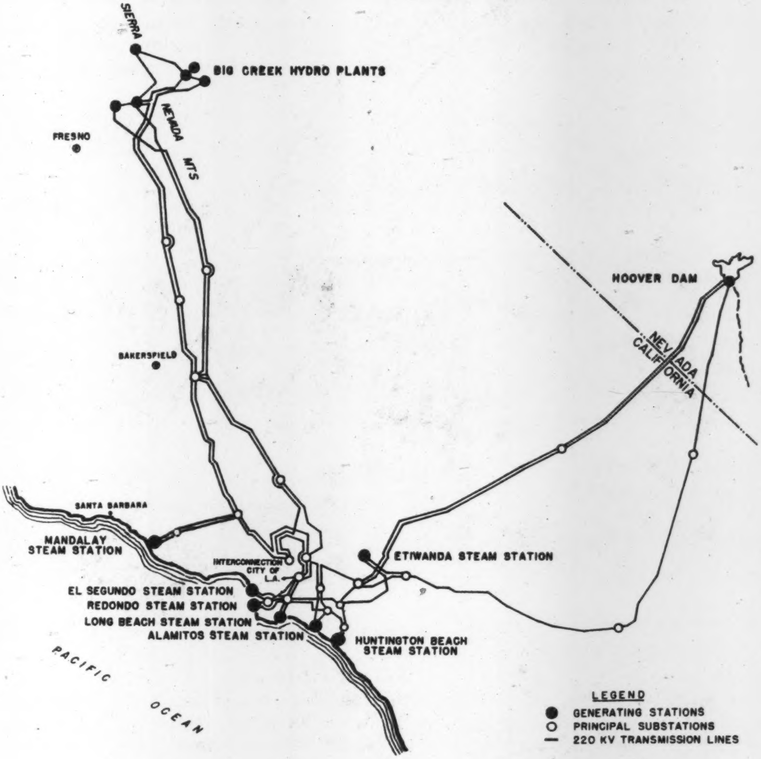
SIERRA
BIG CREEK HYDRO PLANTS
NEVADA MTS
FRESNO
BAKERSFIELD
HOOVER DAM
NEVADA
CALIFORNIA
SANTA BARBARA
MANDALAY STEAM STATION
INTERCONNECTION CITY OF L.A.
ETIWANDA STEAM STATION
EL SEGUNDO STEAM STATION
REDONDO STEAM STATION
LONG BEACH STEAM STATION
ALAMITOS STEAM STATION
HUNTINGTON BEACH STEAM STATION
PACIFIC OCEAN
LEGEND
● GENERATING STATIONS
○ PRINCIPAL SUBSTATIONS
— 220 KV TRANSMISSION LINES

FIG. 1.—SOUTHERN CALIFORNIA EDISON COMPANY MAJOR GENERATING STATIONS

Mandalay.—The first unit at the Mandalay Steam Station is scheduled for operation in March, 1959; consequently no experience has been obtained as of the writing of this paper with the cooling water system at this station. The installation is reviewed at this time because it represents, in some ways, a new cooling water practice for this company. The station is located on Mandalay Beach about 4 miles west of the town of Oxnard and is just south of the mouth of the Santa Clara River (Fig. 2).

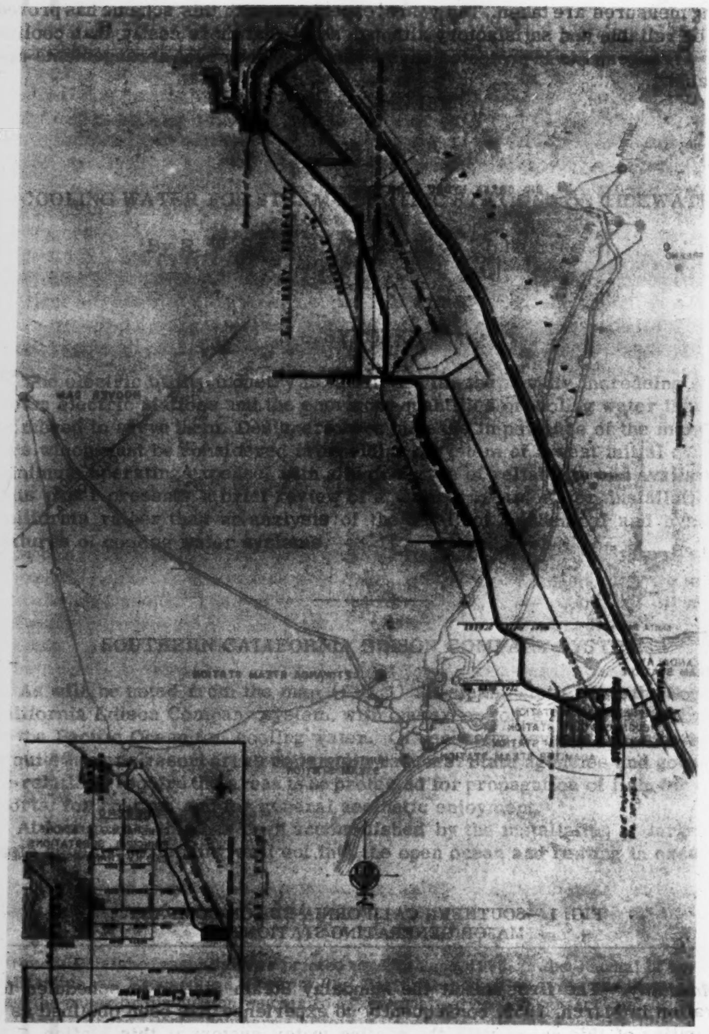

FIG. 2.—MANDALAY STEAM STATION VICINITY MAP

The initial cooling water system study considered the use of conduits extending into the ocean similar to those at Redondo, El Segundo and Huntington Beach. However, the flat sea bottom gradient and the proximity of the mouth of the Santa Clara River to the station site made a long conduit necessary. Also a long record of offshore soundings showed a variation of as much as 10 ft in sand elevation at the proposed intake location. Consideration was then given to the development of a canal intake system from Port Huemene harbor, about 5 miles to the south. Because of problems associated with tidal effects in long canals, a scale model of the proposed canal was built to simulate various operating conditions. This hydraulic model confirmed that a canal could be constructed economically for a design flow of at least 800 cfs, sufficient to support four 200 mw units.

OUTFALL STRUCTURE
No. of Units - 2
Station Capacity
Operating Date

FIG. 3.—MANDALAY STEAM STATION COOLING WATER SYSTEM

The present installation will consist of two 200 mw units. Cost of the canal was about $2,000,000 less than the estimated cost for offshore submarine conduits. Approximately 1,800,000 cu yd of material were excavated in the construction. The canal was built through an agricultural area where shallow ground water level is consistently above sea level, so that there can be no seepage of ocean water into the shallow aquifer. Water is discharged through an outfall structure consisting of a rock-revetted channel extending across the beach to the mean high tide level (Fig. 3). Cost of intake canal and rock-revetted discharge channel was about $2,000,000.

El Segundo.—This station is situated in the city of El Segundo, on the ocean front. It is at the foot of sand dune bluffs and about 300 ft from the shore line. There are two units with a total capacity of 350,000 kw. The first was placed in commercial operation in 1955 and the second in 1956. The cooling water system is typical of several other of the company's installations with two 10 ft inside-diam precast concrete conduits extending 2,600 ft and 2,100 ft along the ocean floor into the Pacific Ocean for the intake and discharge.

Terminal structures are located well beyond the area of surf disturbance in 30 ft of water at low tide. The water flows by gravity to the intake and screen structure on the site. By referring to Fig. 4, the proximity of the intake structure to the station proper can be seen. Total water flow is about 320 cfs. During the early months of operation fish swam into and concentrated in the intake fore-bay, causing some plugging of the traveling screens. Development and installation of the velocity cap has solved this problem.[3] Construction cost of the two marine conduits up to the intake structure was $2,600,000.

Long Beach.—The Long Beach Station is the oldest steam station on the company's system, now containing units installed in 1924, with additions made in 1926, 1928, 1943 and 1948. The total capacity is 400,000 kw. It is situated on Terminal Island near Long Beach on a site favorable located for obtaining a large supply of cooling water from the adjacent ship channel at relatively small expense.

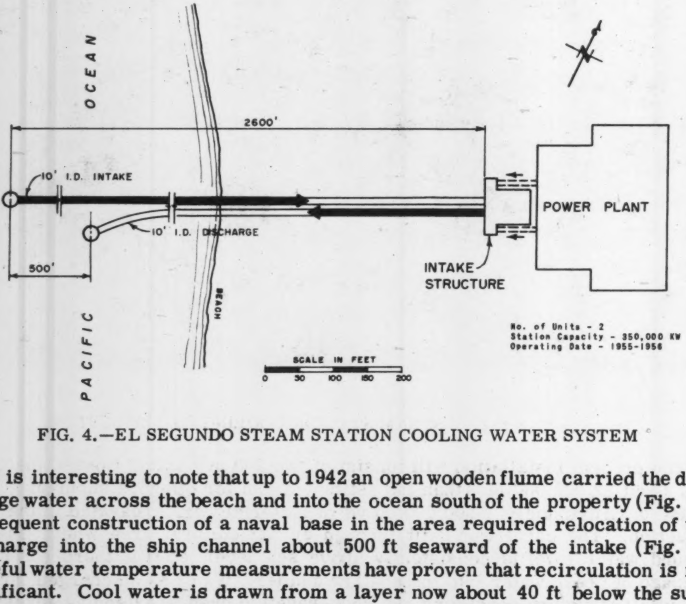

FIG. 4.—EL SEGUNDO STEAM STATION COOLING WATER SYSTEM

It is interesting to note that up to 1942 an open wooden flume carried the discharge water across the beach and into the ocean south of the property (Fig. 5). Subsequent construction of a naval base in the area required relocation of the discharge into the ship channel about 500 ft seaward of the intake (Fig. 6). Careful water temperature measurements have proven that recirculation is not significant. Cool water is drawn from a layer now about 40 ft below the surface at low tide and tidal action removes the warmed surface layer.

Redondo.—The Redondo Beach Steam Station is located on the water front near the northern limits of the city of Redondo Beach. The station consists of two separate plants, with separate but interconnected cooling water facilities common to both. There are six generating units with a total capacity of 638,000 kw. (Fig. 7). The first unit was placed in service in 1948 and the sixth unit in

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[3] "Ocean Cooling Water System for 800 MW Steam Power Station," by R. H. Weight, Proceedings, ASCE, Vol. 84, No. PO6, December, 1958.

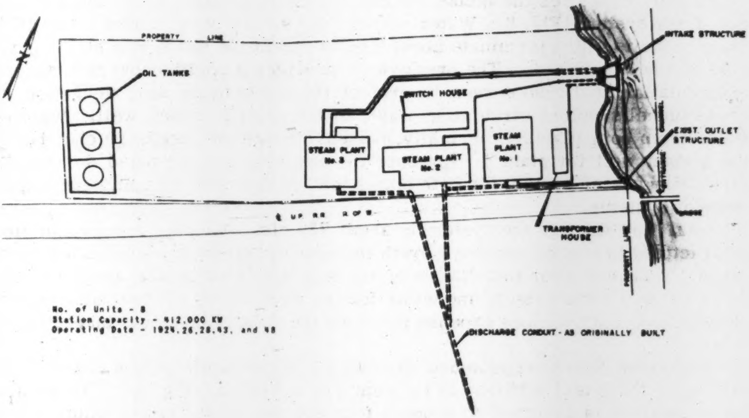

N
PROPERTY LINE
OIL TANKS
SWITCH HOUSE
INTAKE STRUCTURE
STEAM PLANT No. 3
STEAM PLANT No. 2
STEAM PLANT No. 1
EXIST. OUTLET STRUCTURE
CHANNEL
ENTRANCE
BRIDGE
℄ UP RR R OF W
TRANSFORMER HOUSE
PROPERTY LINE
No. of Units - 8
Station Capacity - 412,000 KW
Operating Date - 1924,26,28,43, and 48
DISCHARGE CONDUIT - AS ORIGINALLY BUILT

FIG. 5.—LONG BEACH STEAM STATION COOLING WATER SYSTEM

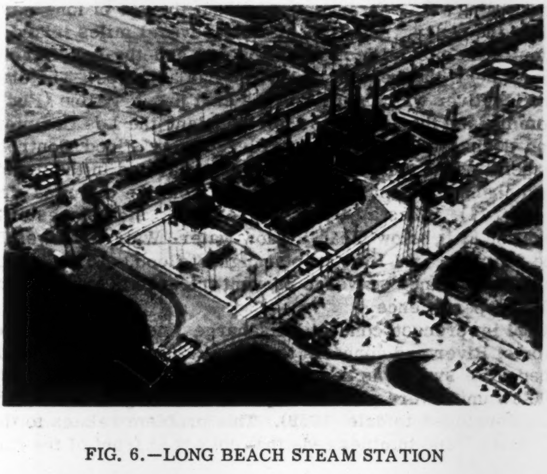


FIG. 6.—LONG BEACH STEAM STATION

1957. Circulating water enters through underwater vertical entrance type chambers located approximately 1,700 ft offshore south of the yacht basin breakwater, and flows to the station by gravity through two 10 ft diam reinforced concrete pipe conduits buried beneath the ocean floor.

Velocity caps over the intake conduit entrance chamber aid in keeping fish out of the system (Fig. 8). Water is returned to the ocean through two 10 ft diam conduits which terminate about 2,000 ft from the shore line on the north side of the breakwater. The breakwater provides a positive barrier against recirculation, and also serves to protect the intake from sand accretion. A cross tunnel supplies circulating water to the plant 1 screen well. Warmed water from both plants is normally passed through the discharge chamber at the screen well for plant 1. However the conduits are arranged so that the circulating water system for both plants may be operated as completely independent systems.

Total flow through the system is about 720 cfs. Periodic reversal of flow is practiced to control marine growth through the effect of elevated temperatures. Total costs for installation of the four marine conduits alone amounts to $4,400,000. The system including fouling control has been so satisfactory that the only maintenance expense has been the cost of an annual inspection by a diver.

Huntington Beach.—Located on the coast 30 miles south of Los Angeles, this station is the latest addition to the company's system (Fig. 9). The cooling water system is designed to support four 200 mw units, two of which are in service. Operation of the first unit started in June, 1958. Maximum design water flow for four units is 792 cfs. Except for size the cooling water system follows the same general design used at Redondo and El Segundo.

14-ft internal diam intake and discharge conduits extend about one-half mile into the ocean with about 500 ft separation between the terminal points to minimize recirculation. Construction of the marine conduits cost about $4,100,000.

Alamitos.—The Alamitos Steam Station of the Southern California Edison Company is of interest because the excellent inland site location permitted an inexpensive cooling water arrangement without the use of long marine conduits extending into the ocean. Although situated several miles inland, it is a tidewater station. The site utilizes Alamitos bay and the Los Cerritos Drain, a storm channel near the west property line, as a source. The discharge is to the San Gabriel River which parallels the east property line (Fig. 10).

To accommodate the large flow to the plant the Los Cerritos Drain was widened and deepened by the Los Angeles County Flood Control District at Edison's expense. The short intake canal is excavated in the natural soils and is an unlined channel except for the turnoff from the Cerritos drain which is riprapped (Fig. 11). It has a bottom width of 56 ft, side slopes of 2-to-1 and a bottom elevation 11 ft below mean low low water. Water at lowest tide is estimated to be 5 ft deep, and 16 ft deep at highest flood. A floating plastic boom across the canal entrance serves to prevent entrance of surface debris and also supports the property fence.

A single 96 in. precast concrete discharge line penetrates the dike along the San Gabriel River and connects to the outfall structure which is flush with the river bed. The system is designed to support a 1,000,000 kw development. Two 175,000 kw units were placed in operation during 1956 and 1957. Only one problem has developed to date (1959). This problem relates to the quantities of debris, "moss," and tumbleweeds that collect in front of the floating boom.

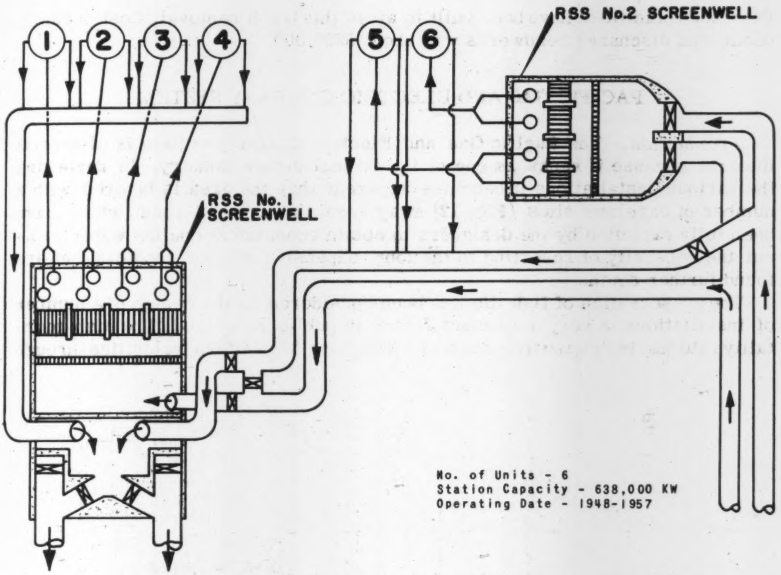

RSS No.2 SCREENWELL
1
2
3
4
5
6
RSS No. 1 SCREENWELL
No. of Units - 6
Station Capacity - 638,000 KW
Operating Date - 1948-1957

FIG. 7.—REDONDO STEAM STATION COOLING WATER SYSTEM

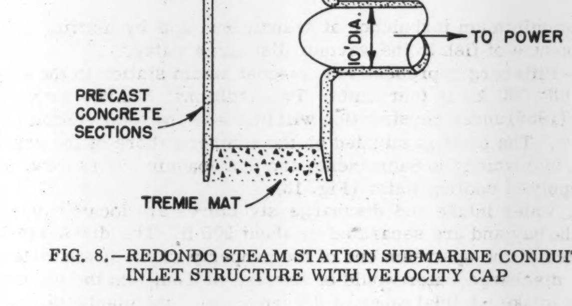

VELOCITY CAP
ELEV. 20' BELOW MEAN LOWER LOW WATER APPROX.
OCEAN FLOOR
ROCK RIP RAP
14' DIA.
10' DIA.
TO POWER PLANT
PRECAST CONCRETE SECTIONS
TREMIE MAT

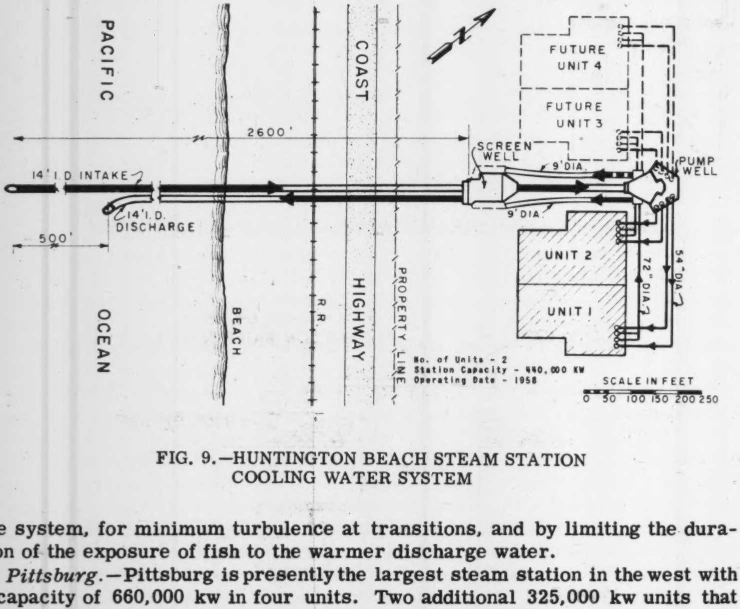
FIG. 8.—REDONDO STEAM STATION SUBMARINE CONDUIT INLET STRUCTURE WITH VELOCITY CAP

Permanent facilities have been built to aid in this trash removal. Cost of canal, boom, and discharge headworks was about $250,000.

PACIFIC GAS AND ELECTRIC COMPANY SYSTEM

Introduction.—The Pacific Gas and Electric Company system is of special interest because it ranks as one of the largest in this country. In reviewing the various installations it becomes apparent that the area is favored with a number of excellent sites (Fig. 12) away from the exposed coast which have been fully exploited by the designers to obtain economical cooling water without the necessity of resorting to the long, expensive marine conduits that are found further south.

The preservation of fish life has been considered, in the design of a number of installations, a very important factor in public relations. The fish mortality rate has been greatly reduced by designing for uniform velocities through the system, for minimum turbulence at transitions, and by limiting the duration of the exposure of fish to the warmer discharge water.



No. of Units - 2
Station Capacity - 440,000 KW
Operating Date - 1958

FIG. 9.—HUNTINGTON BEACH STEAM STATION COOLING WATER SYSTEM

Pittsburg.—Pittsburg is presently the largest steam station in the west with a capacity of 660,000 kw in four units. Two additional 325,000 kw units that are presently (1959) under construction will increase the total station capacity to 1,310,000 kw. The plant is situated on the southern shore of the upper part of Suisun Bay, into which the Sacramento and San Joaquin rivers flow, providing a large supply of cooling water (Fig. 13).

Circulating water intake and discharge structures are located directly on the shores of the bay and are separated by about 900 ft. The discharge outfall has been located at the upstream end of the property where it was determined that the warm discharge water would be carried well out into the main stream and bypass the intake. A tidal range of 8.5 ft occurs at the plant site, and water temperatures range from 47° to 73°.

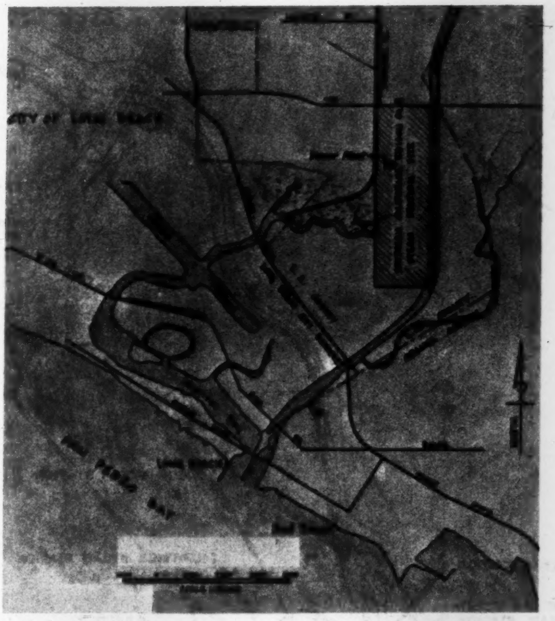

CITY OF LONG BEACH
SAN PEDRO BAY

FIG. 10.—ALAMITOS STEAM STATION VICINITY MAP

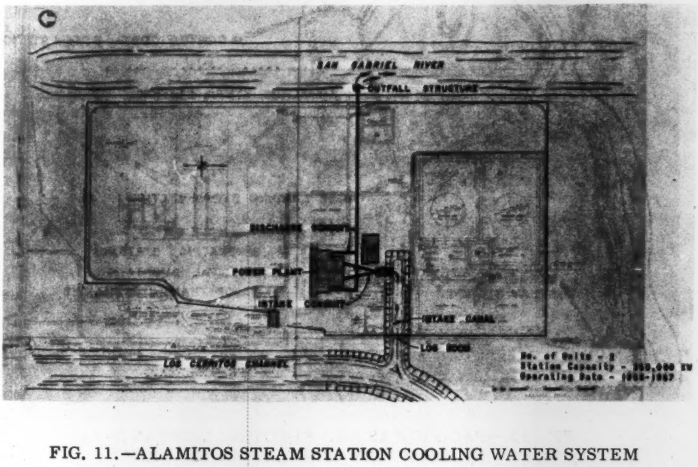

SAN GABRIEL RIVER
OUTFALL STRUCTURE
DISCHARGE CONDUIT
POWER PLANT
INTAKE CONDUIT
INTAKE CANAL
LOG BOOM
LOS CERRITOS CHANNEL
No. of Units - 2
Station Capacity - 350,000 KW
Operating Date - 1956-1957

FIG. 11.—ALAMITOS STEAM STATION COOLING WATER SYSTEM

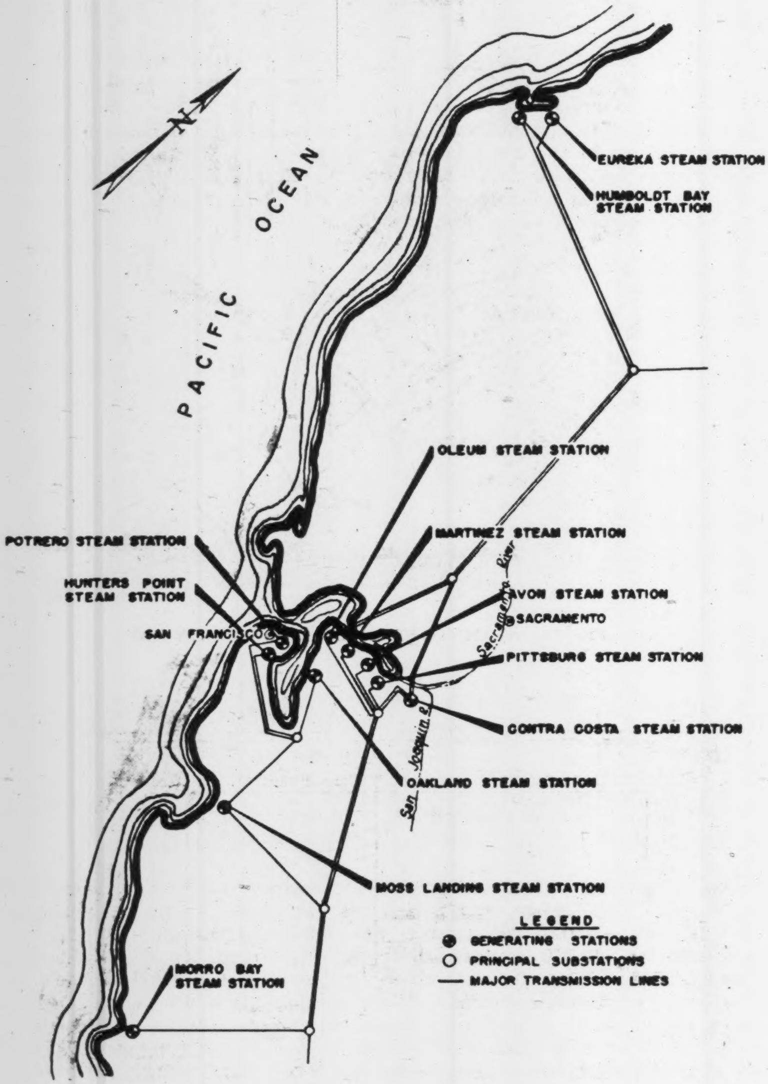

N
PACIFIC OCEAN
EUREKA STEAM STATION
HUMBOLDT BAY STEAM STATION
OLEUM STEAM STATION
POTRERO STEAM STATION
MARTINEZ STEAM STATION
HUNTERS POINT STEAM STATION
AVON STEAM STATION
SACRAMENTO
SAN FRANCISCO
Sacramento River
PITTSBURG STEAM STATION
San Joaquin R.
CONTRA COSTA STEAM STATION
OAKLAND STEAM STATION
MOSS LANDING STEAM STATION
LEGEND
● GENERATING STATIONS
○ PRINCIPAL SUBSTATIONS
— MAJOR TRANSMISSION LINES
MORRO BAY STEAM STATION

FIG. 12.—PACIFIC GAS AND ELECTRIC COMPANY MAJOR STEAM GENERATING STATIONS

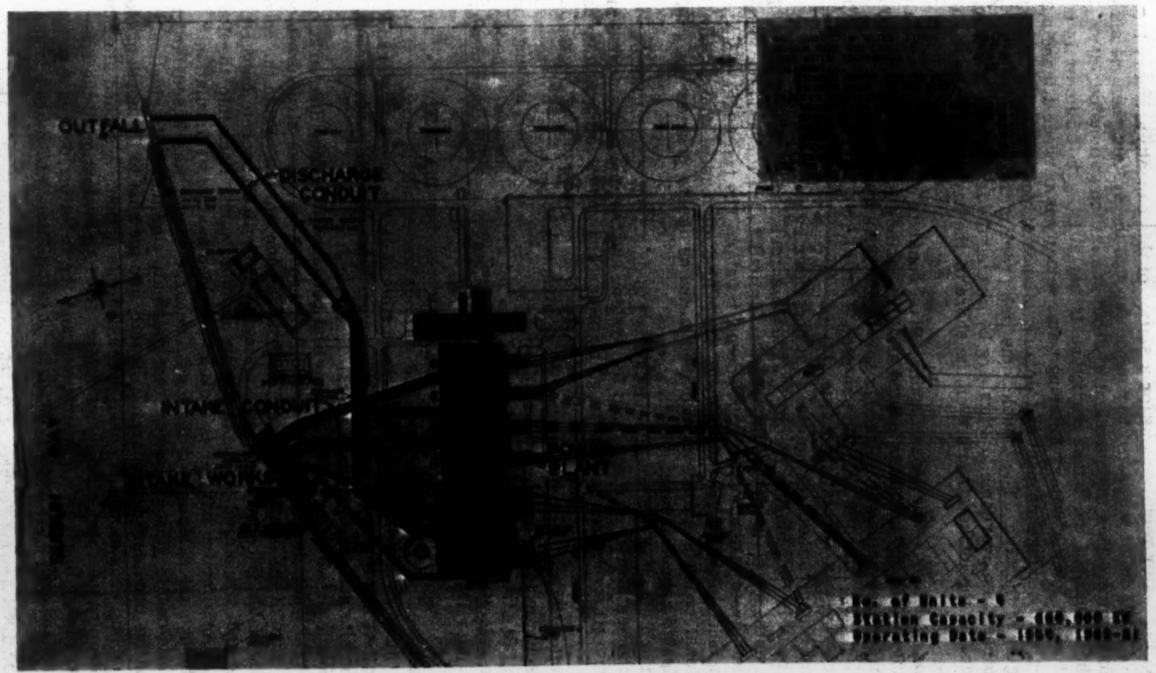

OUTFALL
DISCHARGE CONDUIT
INTAKE CONDUIT
INTAKE WORKS
PLANT
No. of Units -
Station Capacity -
Operating Date -

FIG. 13.—PITTSBURGH STEAM STATION COOLING WATER SYSTEM

Cooling water requirements for a station of this capacity are very large. A total of 1,622 cfs of water will be pumped through the system when the six units are in service. This is equivalent to about one billion gallons per day.

The bottom of the intake structure is set at El. -15 ft 5 in. or 14 ft 3 in. below mean low low water. Entering water velocity is between 1.5 and 2.0 fps. Silting of the intake channel and floating islands of peat requires occasional dredging of the area. The troublesome fish problem that developed at Contra Costa has been overcome at Pittsburg by placing the trash rack and traveling screens at the intake structure on the shore of the bay instead of close to the plant.

Moss Landing.—The Moss Landing Plant consists of five units with a total capacity of 575,000 kw and is located at Moss Landing Harbor in Monterey Bay. The first three units were placed in service in 1950. Two additional units were added in 1952. The site selection on protected water has permitted a very compact cooling water installation (Fig. 14).

Water enters the intake on the east and flows by gravity through two 9 ft 4 in. by 10 ft 7 in. tunnels about 350 ft to the screen and pump structure. Flow at the entrance of the intake structure is limited to the lower 11 ft. This is, presumably, to take advantage of the colder water. The two intake tunnels are not included in the recirculation system provided for mussel control; however, additional cross section was provided in the design to provide margins for anticipated normal mussel growth.

Manual cleaning has not been necessary to date. On occasion large schools of anchovies have been drawn into the system and became concentrated in the screen well structure, causing plugging of screens and affecting station capacity. Note that in the design of later stations this problem was overcome by combining the screen well with the intake structure.

Discharge is to the north into Elkhorn Slough. A lip at the 'leaving end' of the outfall structure tends to direct the flow toward the surface. The low entrance velocities at the intake have not caused any noticeable sand movement and to date dredging has not been necessary.

Morro Bay.—The Morro Bay Station is situated on the Pacific Ocean, half way between San Francisco and Los Angeles and 13 miles northwest of San Luis Obispo. The plant site is at the north end of the bay near Morro Rock. The initial installation of two 156,250 kw units was placed in service in 1955, and the site can accommodate a total of eight units.

The cooling water system is of interest because of the long discharge tunnel (Fig. 15). Water is taken from Morro Bay through an intake structure located at the edge of the bay and pumped 700 ft to the condensers. From the condensers the water is conveyed through a single 7 ft by 10 ft concrete culvert 2,850 ft long and discharged on the northerly side of Morro Rock into the Pacific Ocean. An effective barrier is thereby provided against recirculation.

Although the selection of the long discharge tunnel was based largely on economic considerations, other factors also affected the final selection.[4] One alternative considered discharging directly into the surf to the north of Morro Rock. Clam beds along the coast precluded this action. A second alternative was to return the water directly into Morro Bay. An evaluation of the estimated increase in cooling water temperatures and the resulting decrease in turbine capacity and efficiency proved the present installation to be more economical.

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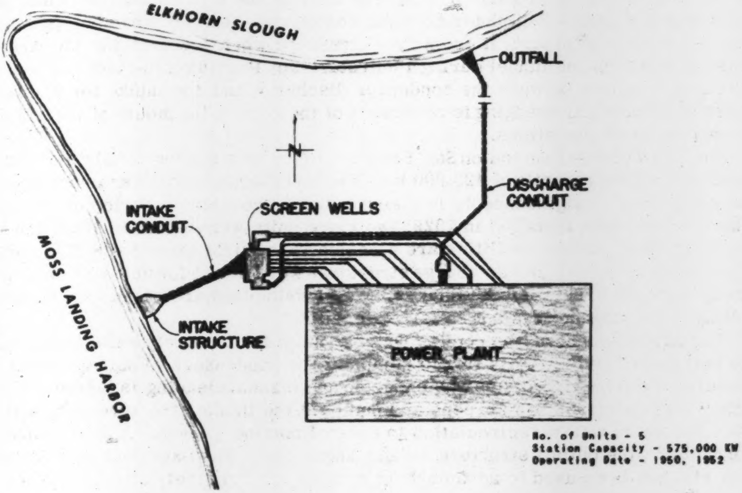
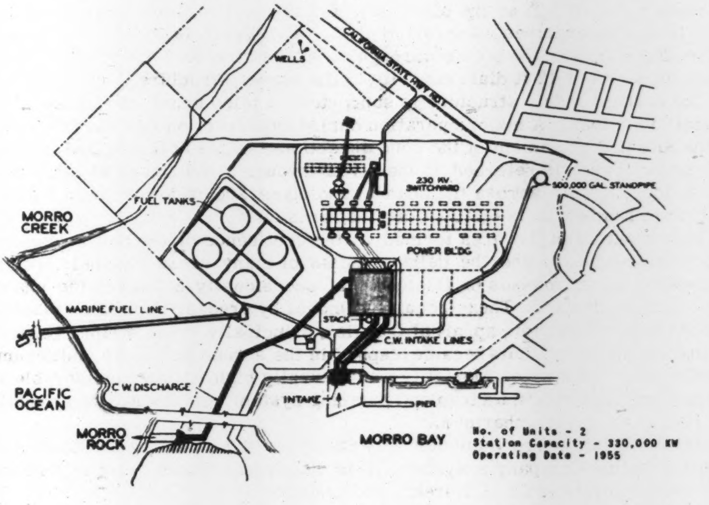
[4] "Civil Engineering Features of the Morro Bay Station," by J. G. Thon and Gordon L. Coltrin, Transactions, ASCE, Vol. 123, 1958, p. 207.


ELKHORN SLOUGH
OUTFALL
N
DISCHARGE CONDUIT
SCREEN WELLS
INTAKE CONDUIT
MOSS LANDING HARBOR
INTAKE STRUCTURE
POWER PLANT

No. of Units - 5
Station Capacity - 575,000 KW
Operating Date - 1950, 1952

FIG. 14.—MOSS LANDING STEAM STATION COOLING WATER SYSTEM


WELLS
CALIFORNIA STATE HWY 1
230 KV SWITCHYARD
500,000 GAL. STANDPIPE
FUEL TANKS
MORRO CREEK
POWER BL'DG
MARINE FUEL LINE
STACK
C.W. INTAKE LINES
C.W. DISCHARGE
PACIFIC OCEAN
INTAKE
PIER
MORRO ROCK
MORRO BAY

No. of Units - 2
Station Capacity - 330,000 KW
Operating Date - 1955

FIG. 15.—MORRO BAY STEAM STATION COOLING WATER SYSTEM

The possible effect of higher water temperature in the bay on an oyster industry in the vicinity was also a factor. The elevation of the long discharge tunnel is such that it remains full under all tidal conditions. A surge chamber is built into the system to protect it from over-pressure when accelerating the large mass of water in the tunnel during a unit start-up. Provision is made for periodic recirculation between the condenser discharge and the intake for mussel control. Occasional dredging is necessary of the area at the mouth of the intake to remove sand accretions.

Hunters Point.—Located on San Francisco Bay, this station consists of four units with a total capacity of 420,000 kw. The installation affords an opportunity for comparing design concepts in a single station covering a period of 29-yr. The first unit was installed in 1929; two more units were added in 1948, and a fourth in 1958. Intake facilities are common to the first three units (Fig. 16), while separate intake and discharge structures were added for unit 4. A channel dredged to -14 ft serves both intakes. Water velocities are below one fps and silting of the channel has not so far materialized.

The arrangement of the cooling water system for the early units is typical for that period, with gravity flow to pumps at the condensers. Fouling of intake tunnels by marine organisms was expected and manual cleaning is routine. The unit installed in 1958 has screens and pumps at the intake structure with facilities for warm water recirculation to control marine growth. A short tunnel connects to the outfall structure on the shore line. The excellent location of this site has been used to advantage in designing a relatively simple and economical cooling water installation.

Contra Costa.—The first three units of this large, modern station were placed in operation in 1951. Two additional units were added in 1953, giving a total capacity of 575,000 kw. The site is situated near the mouth of the San Joaquin River, 2-1/2 miles east of Antioch. Tidal conditions prevail, with an extreme range of 8 ft at the plant site. Cooling water requirement is 863 cfs and river temperatures range from 45° to 74°. Water is withdrawn from the river (Fig. 17) near the bottom through an intake structure 410 ft offshore and flows through two 12 ft diam conduits to the screen structure.

The entire conduit structure is supported on piles under the intake and at conduit junctions. A major operation during construction was the fabrication at the site and placement of the conduit sections, each 100 ft long and weighing 480 tons. Water is returned to the river through a discharge channel about 250 ft long; a weir across the channel and about 200 ft downstream from the discharge headworks is effective against erosion.

The intake is in the area formed by the confluence of the San Joaquin and Sacramento rivers, and the delta consists of many cross channels and tule marshes. Large masses of floating peat occasionally collect in the channel and become a hazard. These require removal by dredge. Soon after plant operation began it became apparent that fish, principally young striped bass, entering the intake conduits became trapped in the screen structure and accumulated in large quantities. Mortality was naturally high. After considerable research and investigation a trap and pumping system was devised which returns the fish to the river unharmed.[5]

Humboldt Bay.—The Humboldt Bay Station is the most recent steam plant addition to this Company's system. It is situated at Buhne Point in Humboldt Bay, seven miles south of Eureka, and consists of two 50,000 kw units. The first of the two units was placed in service in 1956 and the second in late 1958.

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[5] Fish Bulletin No. 92, California Div. of Fish and Game.

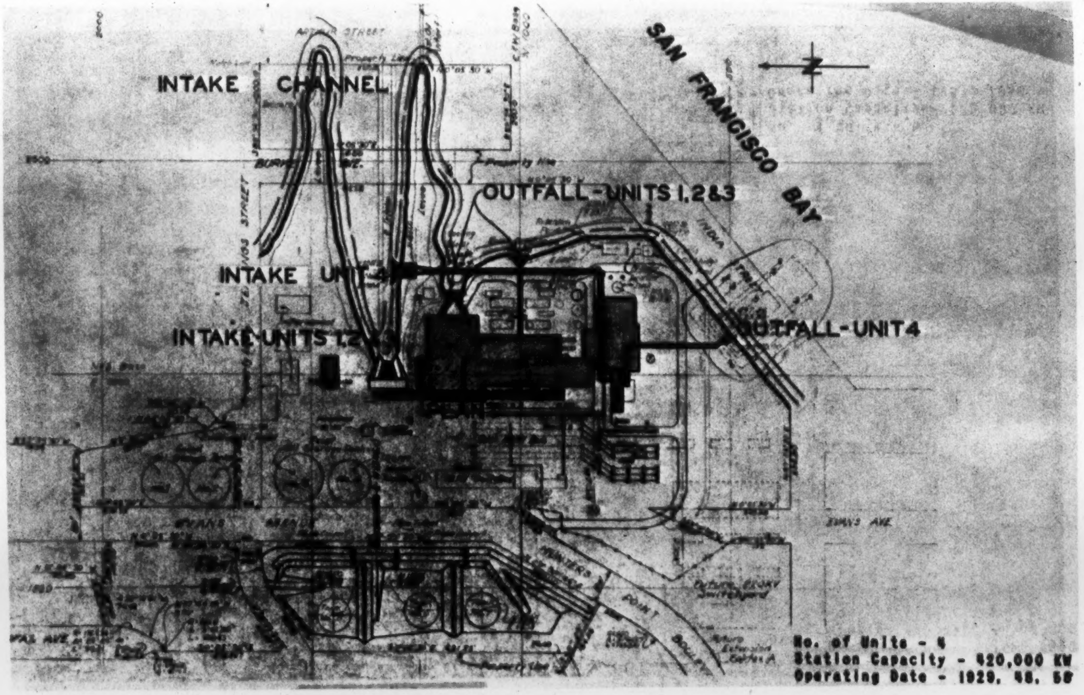

SAN FRANCISCO BAY
N
INTAKE CHANNEL
OUTFALL-UNITS 1,2&3
INTAKE UNIT 4
INTAKE UNITS 1,2&3
OUTFALL-UNIT 4
EVANS AVE.
No. of Units - 4
Station Capacity - 420,000 KW
Operating Date - 1929, 48, 58

FIG. 16.—HUNTERS POINT STEAM STATION COOLING WATER SYSTEM

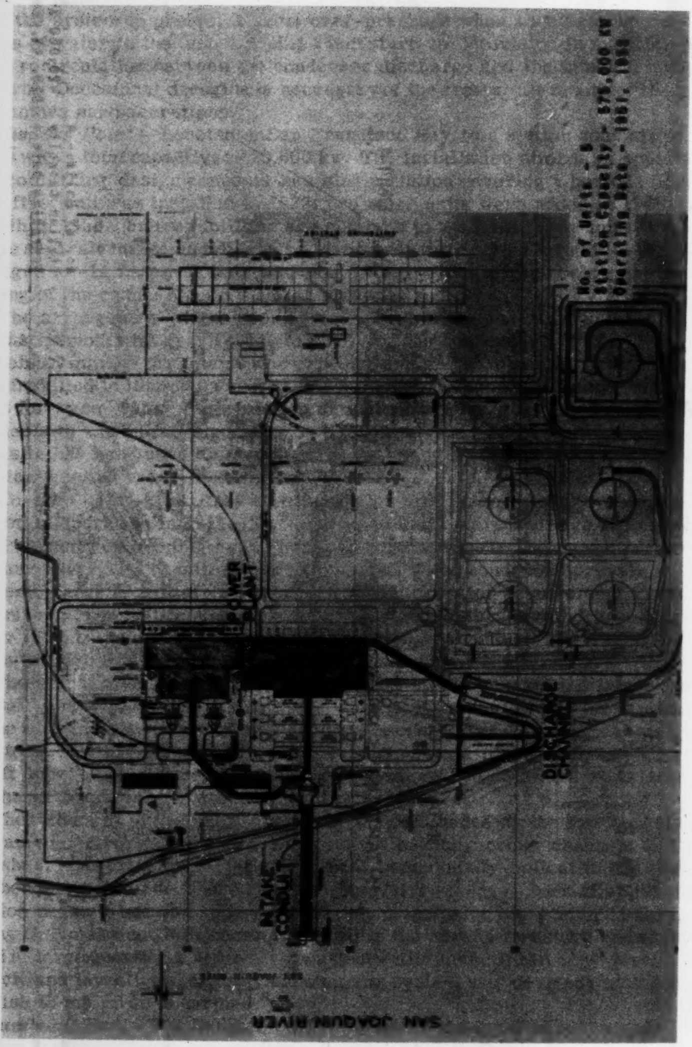

No. of Units - 5
Station Capacity - 575,000 KW
Operating Date - 1951, 1953
POWER PLANT
INTAKE CONDUIT
DISCHARGE CHANNEL
SAN JOAQUIN RIVER

FIG. 17.—CONTRA COSTA STEAM STATION COOLING WATER SYSTEM

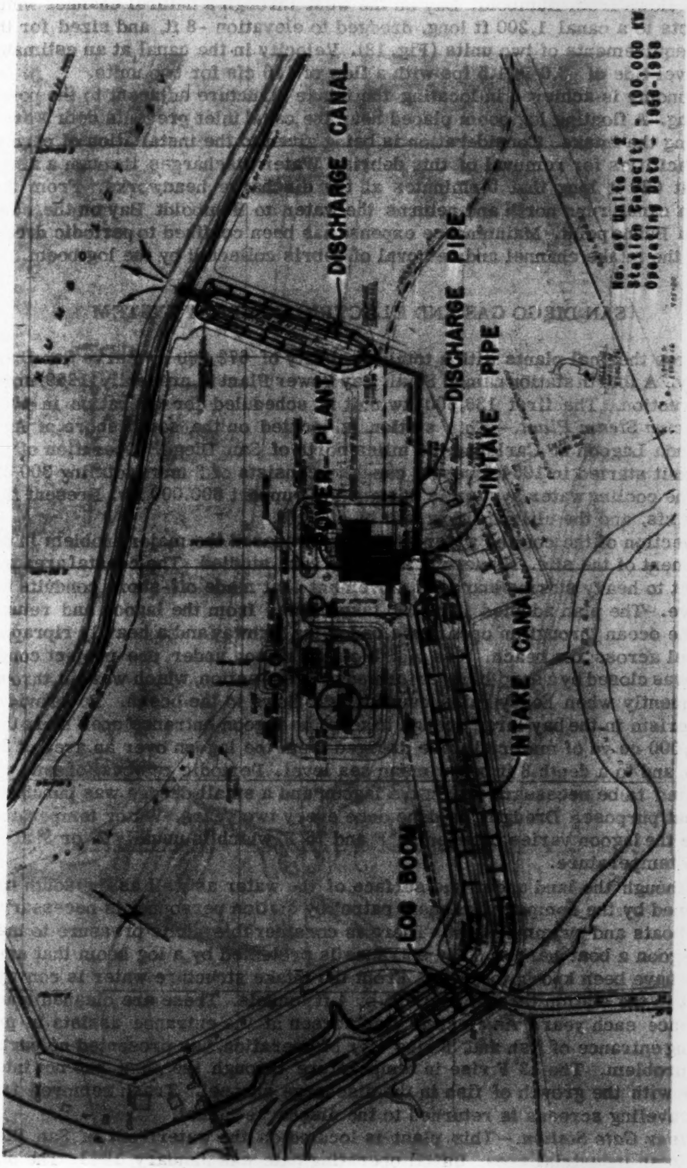

DISCHARGE CANAL
DISCHARGE PIPE
DISCHARGE PIPE
POWER-PLANT
INTAKE PIPE
INTAKE CANAL
LOG BOOM
No. of Units - 2
Station Capacity - 100,000 KW
Operating Date - 1956-1958

FIG. 18.—HUMBOLDT BAY STEAM STATION COOLING WATER SYSTEM

Water flows from Humboldt Bay on the west through a natural channel which connects to a canal 1,200 ft long, dredged to elevation -8 ft, and sized for the flow requirements of two units (Fig. 18). Velocity in the canal at an estimated low level tide of -3.0 is 1.5 fps with a flow of 116 cfs for two units.

Economy is achieved in locating the intake structure adjacent to the power building. A floating log boom placed near the canal inlet prevents debris from entering the intake. Consideration is being given to the installation of permanent facilities for removal of this debris. Water discharges through a single conduit 620 ft long that terminates at the discharge headworks. From this point a canal runs north and returns the water to Humboldt Bay on the other side of Buhne point. Maintenance expense has been confined to periodic dredging of the intake channel and removal of debris collected by the log boom.

SAN DIEGO GAS AND ELECTRIC COMPANY SYSTEM

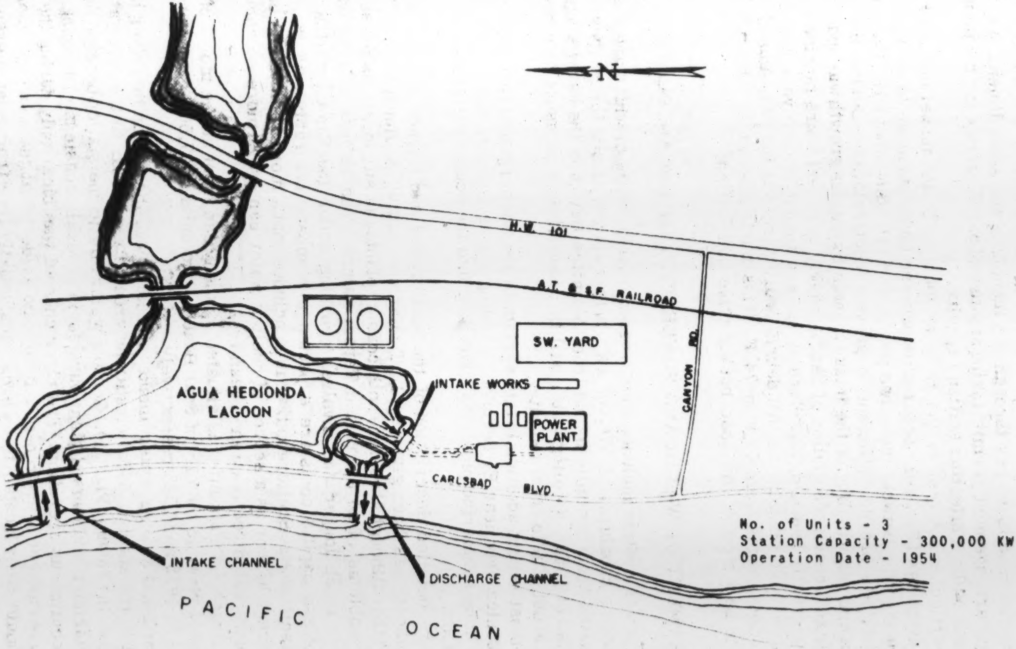
Three thermal plants with a total capability of 672,000 kw serve San Diego County. A fourth station named South Bay Power Plant is presently (1959) under construction. The first 136,500 kw unit is scheduled for operation in 1960.

Encina Steam Plant.—This station is located on the south shore of Agua Hedionda Lagoon at Carlsbad, 34 miles north of San Diego. Operation of the first unit started in 1954; present capacity consists of 3 units totaling 300,000 kw. The cooling water system is designed to support 600,000 kw. Present flow is 330 cfs, and the ultimate flow will be 600 cfs.

Selection of the cooling water arrangement posed the major problem in development of the site. Numerous schemes were studied. The coastal area was subject to heavy storms and sand movement that made off-shore conduits expensive. The plan adopted (Fig. 19) takes water from the lagoon and returns it to the ocean through an open flume under the highway and a heavily riprapped channel across the beach. The mouth of the lagoon under pre-project conditions was closed by a sand barrier formed by ocean action, which was cut through infrequently when heavy rains caused high flows to the ocean. To provide a tidal prism in the bay large enough to keep the lagoon entrance open more than 4,000,000 cu yd of material were dredged from the lagoon over an area of 240 acres, and to a depth 8 ft below mean sea level. Periodic removal of sand was expected to be necessary within the lagoon and a small dredge was purchased for that purpose. Dredging is done once every two years. Water temperature inside the lagoon varies between 54° F and 76° F, which is usually 2° or 3° above ocean temperature.

Although the land under the surface of the water as well as the south bank is owned by the company, frequent patrol by station personnel is necessary to keep boats and swimmers out. There is considerable public pressure to make the lagoon a boat harbor. The entrance is protected by a log boom that small boats have been known to cross. From the intake structure water is conveyed 520 ft to the station through two 8 ft by 4 ft tunnels. These are cleaned manually once each year. An electric fish screen at the entrance assists in preventing entrance of fish and, during 4 yr of operation, has presented no particular problem. The 13° F rise in temperature through the plant has not interfered with the growth of fish in the discharge channel. Trash removed from the traveling screens is returned to the discharge tunnel.

Silver Gate Station.—This plant is located on the waterfront of San Diego Bay, in an industrial area. Initial operating date was January, 1943. The plant


N
H.W. 101
A.T. & S.F. RAILROAD
SW. YARD
CANYON RD.
INTAKE WORKS
POWER PLANT
AGUA HEDIONDA LAGOON
CARLSBAD BLVD.
INTAKE CHANNEL
DISCHARGE CHANNEL
PACIFIC OCEAN

No. of Units - 3
Station Capacity - 300,000 KW
Operation Date - 1954

FIG. 19.—ENCINA STEAM STATION COOLING WATER SYSTEM

has four units, with a total capacity 215,000 kw. The cooling water flow is 333 cfs. Water is conveyed to the station through two 8 ft by 8 ft tunnels 760 ft long, and returned to the bay through two tunnels of the same dimensions. This duplicate arrangement permits taking any tunnel out of service for cleaning and inspection without affecting station capacity.

Of particular interest is the relative location of the tunnel inlet and discharge openings (Fig. 20). Separation between these openings is about 200 ft. Steel sheet piling between these two points, and extending about 100 ft from the shoreline, has proved sufficient to prevent recirculation. During the early years of operation fouling of the intake tunnels by marine growth was extensive. However as a result of pollution of San Diego Bay in recent years marine growth has been greatly reduced, and flushing of each tunnel twice a year now maintains reasonable cleanliness. All debris is discharged to the bay. Annual water temperature ranges from 61° F to 74° F. This cooling water arrangement has proved to be a simple, low cost, but effective installation.

DEPARTMENT OF WATER AND POWER, CITY OF LOS ANGELES SYSTEM

The power system includes four thermal plants, three of which are coastal plants with a capability of 576,000 kw. A new station named the Haynes Plant and situated about one mile inland from Alamitos Bay is in the early stages of design (as of 1959). Cooling water will be drawn from a pleasure boat harbor located about a mile from the site.

The most recent addition to this system is the Scattergood Plant located on Santa Monica Bay near El Segundo. Cooling water is conducted to the plant through long submarine pipelines extending into the ocean and resting on the ocean floor.

A more detailed description of the Scattergood and Haynes cooling water arrangements has been given by L. T. Mariner and W. A. Hunsucker.[6] They point out that although these are both coastal plants the site locations presented design problems of an entirely different nature. The Haynes cooling water system is much longer and involves crossing under the San Gabriel River. Nevertheless capitol costs are expected to compare favorably with the submarine facilities at Scattergood. It is concluded that a more remote site using an improved harbor as a source of cooling water can prove more economical than a plant site directly on the ocean front.

Seal Beach.—This station is situated on the ocean front at the mouth of the San Gabriel river in the city of Seal Beach. Two units of 37,500 kw each were installed in 1925 and 1928 respectively.

The original cooling water intake structure was located on the river bank adjacent to the station. Tidal conditions created a sand bar in the river at the entrance to the intake (Fig. 21). Cooling water flow to the station became seriously restricted and caused plant outages on several occasions. Dredging became a continuous operation. The river channel was clear only during the infrequent periods of heavy run-off. It became evident that the source of cooling water should be relocated to an area substantially free from sand movement. A levee was constructed on the opposite side of the river at the entrance to Alamitos Bay. From the original intake structure two 8 ft-diam conduits extend under the river bed, penetrate the levee and terminate in a new intake

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[6] "Ocean Cooling Water Systems for Two Thermal Plants," by L. T. Mariner and W. A. Hunsucker, Proceedings, ASCE, Vol. 85, No. PO4, 1959.

FIG. 20.—SILVER GATE STEAM STATION

FIG. 21.—SEAL BEACH STEAM STATION

structure on Alamitos Bay (Fig. 22). The levee provides an area of comparatively calm water for the intake and forms an effective barrier against sand movement in the river; discharge is to the San Gabriel river.

Harbor.—The station is situated at the Los Angeles Harbor in Wilmington and consists of five units with a total capacity of 355,000 kw. The first unit was placed in service in 1943. Cooling water is taken from slip No. 5 and flows through two 8 ft-diam conduits 1,355 ft long to the screen and pump structure at the station (Fig. 23). Discharge is to the West Basin.

Water pollution, prevalent in the area, is caused by adjacent fish canneries and from ships docked in the harbor. The intake conduits were designed originally for 550 cfs. Later additions of larger units increased the flow to 700 cfs. Drawdown, produced in the intake tunnels during operation with all pumps in service and low tide conditions, can approach minimum pump submergence requirements, particularly if fouling of tunnels becomes acute. Tunnel cleaning is normally performed three times each year.

CONCLUSIONS

The general review of the cooling water systems of the 18 steam stations briefly summarized in this paper leads to the following comments:

1. Although the availability of cooling water has been a major factor in the selection of plant sites, the designer is usually faced with making a choice of several alternate cooling water arrangements after the site is purchased. Economic considerations usually prevail, but they are not necessarily paramount. Public relations and preservation of fish life and other natural resources must also be recognized.
2. There has been a steady improvement in the design of cooling water systems over the past 30 yr as a result of a better understanding of operating and maintenance problems.
3. Desirable station sites are becoming difficult to obtain in many areas, thus emphasizing the need for maximum utilization of existing and future sites.
4. A variety of methods have been used successfully in meeting operating problems connected with marine growth, fish preservation and sand deposition. Thermal shock treatment has become a standard procedure for control of mussels and other marine organisms. The fish problem has been greatly minimized by trapping at intakes and screen wells, and by the installation of velocity caps which change the direction of flow entering submerged pipes from vertical to horizontal. The advice of consultants or specialists in the field of marine biology assisted materially in this work. In general each design is tailored to meet the local conditions economically.
5. Problems associated with open ocean intakes as distinguished from intakes and sheltered bays are mainly the following: a) Sand movement or unstable bottom; b) Marine growth control; and c) Problems associated with the destruction of fish or clogging of screens by fish.

The sand problem is a complicated one along the California coast. The quantity of shifting sand varies tremendously with location. Near the Edison company's Mandalay plant it is estimated to be in the order of magnitude of 1,200,000 cu yd per year. Sites to the southeast or downcoast from large stream will be affected by heavy sand travel for a period of years following a large flood. Rapidly eroding coastlines may also contribute large amounts of

No. of Units - 2
Station Capacity - 75,000 KW
Operating Date - 1925, 28

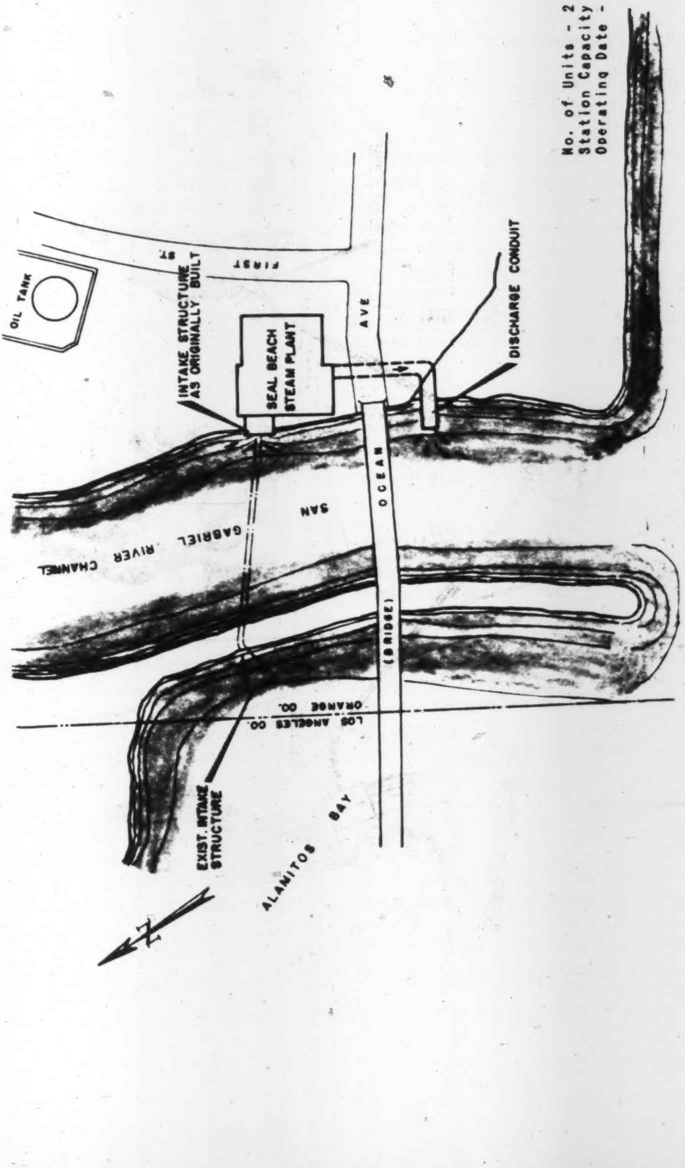
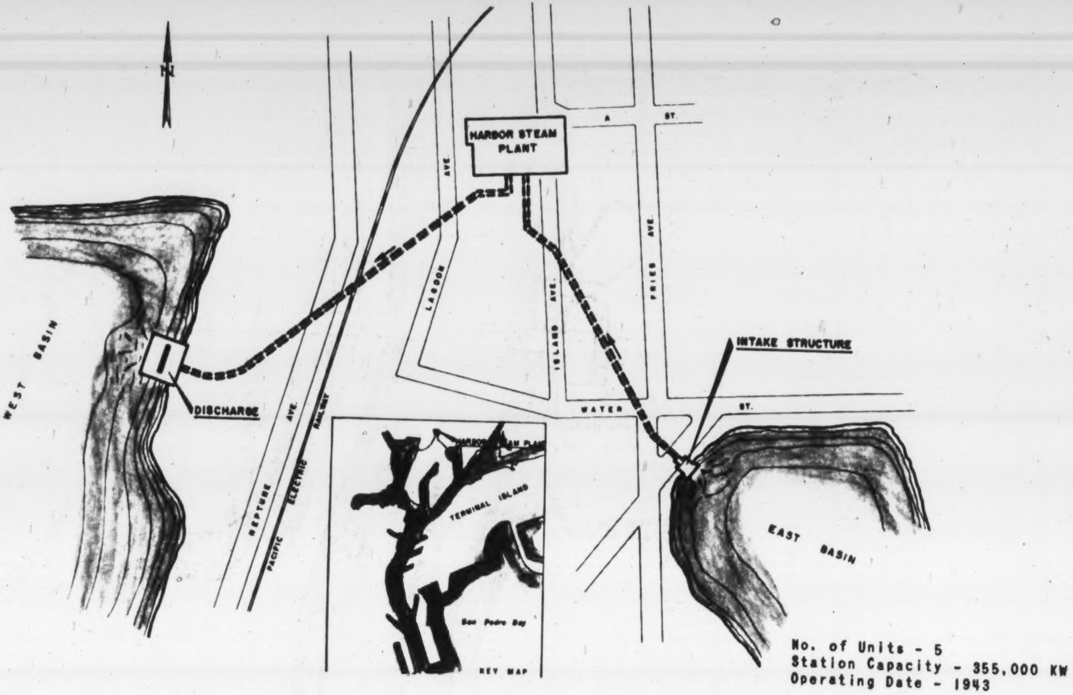

OIL TANK
INTAKE STRUCTURE AS ORIGINALLY BUILT
FIRST ST.
SEAL BEACH STEAM PLANT
AVE
DISCHARGE CONDUIT
SAN GABRIEL RIVER CHANNEL
OCEAN
(BRIDGE)
LOS ANGELES CO.
ORANGE CO.
EXIST. INTAKE STRUCTURE
ALAMITOS BAY
N

FIG. 22.—SEAL BEACH STEAM STATION COOLING WATER SYSTEM



N
HARBOR STEAM PLANT
A ST.
LAGOON AVE.
ISLAND AVE.
FRIES AVE.
WATER ST.
INTAKE STRUCTURE
DISCHARGE
WEST BASIN
EAST BASIN
NEPTUNE AVE.
PACIFIC ELECTRIC RAILWAY
HARBOR STEAM PLANT
TERMINAL ISLAND
San Pedro Bay
KEY MAP

No. of Units - 5
Station Capacity - 355,000 KW
Operating Date - 1943

FIG. 23.—HARBOR STEAM STATION COOLING WATER SYSTEM

sand. At some locations a submarine canyon close to shore acts as a permanent trap to bleed off sand accumulations to deep water, and intakes down-coast from such submarine canyons or rocky promontories are apt to have little trouble with sand. The sand problem is not confined to the surf zone but may extend off shore to water 30 or 40 ft deep.

Construction of groins, training dikes, offshore breakwaters and harbor entrance facilities may have drastic effects on sand problems at intakes depending on the exact relation of the intake to the facility. Because of the complex nature of sand problems it is wise economy to retain consultants who are experts on beach control problems.

6. It can be concluded that the most economical cooling water installation for the larger stations presently in operation is one which draws water from a protected harbor and discharges it into the open ocean.

7. The problems of the future are expected to be of a different order of magnitude. It has been estimated, conservatively, that energy demands will double each 10 years for several decades. This energy will necessarily be supplied by thermal power plants. Regardless of the nature of the fuel, whether it is gas, oil, coal or uranium, tremendous quantities of cooling water will be required. Plants of the future are likely to have capacities of several million kw. Assuming a water use of 1.2 cfs per 1000 kw, cooling water requirements for a 5 million kw plant will be in the order of 6000 cfps. It is evident that few if any natural streams in California can safely provide such quantities of water, and the Pacific Ocean is the logical source.

The alternative of using evaporative type cooling towers for such large plants will probably be uneconomical because of:

a) The large area required for cooling towers and to provide for the optimum arrangement.
b) Difficulties with neighbors resulting from the emission of large volumes of vapor or high humidity air.
c) Consumptive use of about 60,000 acre-ft of water per year for a five million kw plant, modified somewhat by the type of condenser and the steam cycle used.

The effect of recirculation will require careful study. Salt water sites, in California, for these mammoth plants will be restricted to two or three of the larger bays or the open ocean to prevent local warming of the water and a lowering of station efficiency. The capitalized value of 1° F change in cooling water inlet temperature for a 200 mw unit is about $25,000. This figure will increase with larger and more costly units.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the following who have generously contributed material for this paper: John F. Bonner, Gordon L. Coltrin and R. V. Bettinger of Pacific Gas and Electric Company; C. L. Hathaway, E. R. Prout, C. T. Geiger of San Diego Gas and Electric Company; and L. T. Mariner and W. A. Hunsucker of the Department of Water and Power, Los Angeles, Calif.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3176

OPERATION OF A 7-MILE DIGESTED SLUDGE OUTFALL

By Norman B. Hume,[1] F. ASCE, Robert D. Bargman,[2] F. ASCE, Charles G. Gunnerson,[3] F. ASCE, and Charles E. Imel,[4] A. M. ASCE

With Discussion by Messrs. Charles H. Lawrance and David R. Miller; and Norman B. Hume, Robert D. Bargman, Charles G. Gunnerson, Charles E. Imel

SYNOPSIS

Engineering and oceanographic factors that established the design and construction criteria for Los Angeles' digested sludge outfall are reviewed. One year's experience shows that the operation, involving discharge of about 5 mgd of digested sludge and plant effluent into 320 ft of water has fulfilled the expectations of the designers.

INTRODUCTION

Since the inception of the industry the disposal of the solid products of sewage treatment processes has been a bothersome problem. The problem has been intensified by the pressure of rising costs and declining availability of land for historic processes. New materials and construction techniques have opened the door to new extensions of old techniques and made possible disposal practices that were once thought undesirable.

The city of Los Angeles disposes of sewage from the city proper and 16 surrounding municipalities through the Hyperion Treatment Plant. Design

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Note.—Published essentially as printed here, in July, 1959, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2089. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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capacity for the existing plant, placed in operation in 1950, is 245 mgd with present flow being about 270 mdg. Solids disposal methods included digestion, elutriation and mechanical dewatering facilities, utilizing vacuum filtration, and flash drying equipment.

SLUDGE DISPOSAL PRACTICE IN LOS ANGELES

When the plant was designed, it was considered that the revenue received from the fertilizer produced would result in the most economical sludge disposal. In actual practice the fertilizer contains 2% nitrogen, as a result of which disappointing revenues of only $4.00 per ton are obtained. Costs of production for the fiscal year 1956-57 are shown as follows and represent minimum chargeable items:

Production of digested sludge, tons per day		160
Digested sludge removed in mechanical dewatering plant, tons per day		92
Cost per ton solids removed in mechanical dewatering plant:		
	Elutriation	$1.15
	Filtration	6.35
	Drying	4.02
	Fertilizer handling	2.70
	TOTAL	$14.22

Power costs (power consumed is surplus power), fuel charges (digester gas is available), local administrative and general city overhead (including retirement) charges are not included in this table. Such charges would materially increase the cost shown.

The solids escaping capture in the elutriation system were of major concern. Of the 160 tons per day of digested sludge produced, only 92 tons per day were recoverable in the best year of operation and the remaining 68 tons per day escaped to sea via the one-mile ocean outfall. Such disposal was at best undesirable because it was necessary to disinfect the plant effluent using chlorine. In addition, the turbidity of the nearshore waters was significantly increased, and the deposition of solids on the floor of the Bay contributed to the observed effect on bottom dwelling animals. This major process failure has been presented elsewhere in detail.[5]

Air pollution is of major import in the Los Angeles area. It is indeed strange that man sometimes exchanges one pollution problem for another, but this is precisely what happened in this case. Severe limitations of particulate matter emission from any municipal or industrial plant are imposed. A sliding scale is used as a control measure depending on the process weight, (that is, pounds per hour processed), with the upper limit being 40 lb per hr of particulate matter allowed. Average emission of one of the drier units at Hyperion has been about 80 lb per hr. In addition, the stack effluent poses problems of odor, visibility, and so on.

In 1954, Los Angeles was faced with the necessity of expanding facilities consistent with the city's growth. Consulting engineers were utilized by a citizens' committee acting for the city. In view of the problems previously enumerated—costs, solids overflow, and air pollution—it is not strange that alternate means of disposal were sought rather than an increase in mechanical dewatering facilities.

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[5] "Aeration Requirements of a High Oxygen Demand Sewage," by R. D. Bargman, J. M. Betz, and W. F. Garber, *Sewage and Industrial Wastes*, Vol. 29, No. 7, 1957, p. 760.

Our discharge of plant effluent, that already contained a substantial amount of the digested solids to the ocean, pointed to a similar disposal of all digested sludge. This philosophy is not peculiar to the city of Los Angeles, because coastal cities throughout the world discharge their wastes into estuarine or marine waters. In the United States, this may be observed at such places as Boston, Mass., New York, N. Y., the Los Angeles County Sanitation Districts, Oakland, Calif., and Portland, Oreg. The benefits of marine disposal are obvious.

The next question, that of whether to discharge sludge with the effluent or separately, was then considered. While the former method is more common, separate sludge disposal has been practiced by New York City for many years and has been more recently adopted at such installations as Boston's Nut Island Treatment Plant and the paper mill wastes at Port Garnder Bay, Washington.

Probably the most important advantage in utilizing separate disposal of sludge is that of control; the characteristics of the discharge itself can be maintained within reasonable limits. Inherent in the basic recommendations of the Citizens' Committee engineers[6,7] are the following factors:

1. A small diameter sludge line may be economically constructed to such depths at which the sludge discharge will not affect surface waters. Nor will any possible effect on fish be of controlling significance.

2. Sludge disposal is independent from the requirements for additional treatment plant capacity. (It follows that elimination of the sludge from the effluent permits chlorination at reasonable rates in the event that future receiving water uses require disinfection.)

Further, there are advantages from a public relations point of view in that a discharge in deep water far from shore is better than one that can be seen.

To meet the basic requirements for increased capacity at the Hyperion Treatment Plant, the Citizens' Committee followed their consulting engineers, R. R. Kennedy, F. ASCE, of San Francisco, Calif., and Richard D. Pomeroy, F. ASCE, of Pasadena, Calif., in recommending that the treatment be changed so that primary effluent would be discharged 5 miles offshore and digested sludge and supernatant would be discharged 7 miles offshore.

The city subsequently engaged Hyperion Engineers, Inc., a joint venture consisting of the Los Angeles engineering firms of Holmes and Narver, Inc., Koebig and Koebig, and Daniel, Mann, Johnson and Mendenhall. The various

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[6] "Opinions and Comment on Engineering Features of Proposed Solutions of the Sewerage Problem of the City of Los Angeles for Citizens' Committee on Sewerage Problem, George B. Gose, Chairman," by R. R. Kennedy, September, 1954.

[7] "Opinions and Comment on Engineering Features of Proposed Solutions of the Sewerage Problem of the City of Los Angeles for Citizens' Committee on Sewerage Problem, George B. Gose, Chairman," by Richard Pomeroy, September, 1954.

design criteria developed by Hyperion Engineers for the sludge outfall are detailed in their final report[8] and are summarized below.

DESIGN CONSIDERATIONS

The design considerations were as follows:

1. The average (design) sewage flow for the year 2,000 is 420 mgd.
2. The quantity of digested sludge and supernatant will increase from the present 1.32 mgd to 2.45 mgd by the year 2,000. A total capacity of 5.22 mgd is provided in order to allow for a reasonable dilution with final effluent.
3. In order to assure turbulent flow, a minimum velocity of 2.57 fps is required. A velocity of 3.55 fps is provided at the design flow.
4. The design Darcy f is 0.018. While the pumping head is 73 ft, a maximum internal head of 250 ft was provided for.
5. The design life of the pipe is 100 yr. Either concrete or suitably covered steel pipe is satisfactory. The final choice was 22 in. ID steel pipe with a 3/8 in.-wall, a 1/2 in. concrete inner lining, an exterior coating of coal tar, with three wrappings of fiber glass and 1 1/8 in. of reinforced gunite, and with cathodic protection. The weight of the concrete is sufficient to provide negative buoyancy with the pipe full of air as was the case during launching.
6. The required depth at the discharge end was computed by N. H. Brooks,[9,8] M. ASCE, for the condition that the sludge-sea water mixture would remain submerged. Assuming that the discharge has a density of 0.9987 (primary effluent) and mixes with bottom water that is 5°F colder than the surface water and has a density of 1.02589, the mixture will have a density such that the upper surface of a thick effluent field would rise a maximum of 285 ft. The design depth was set at 300 ft (the actual depth is 320 ft) with the discharge at the head of a submarine canyon.
7. Studies of the bottom sediments were made by marine geologists of the Allan Hancock Foundation, University of Southern California[10,8] and of Geological Diving Consultants, Inc., San Diego,[11,8] and by LeRoy Crandall and Associates, consulting foundation engineers, Los Angeles[12,8]. It was found that the bottom consisted of sand and soft silty sand, and that bedrock was not exposed along the alignment.
8. Provision for protection against scour to a depth of 10 ft by wave action along the coast is necessary.[13,8]
9. Analysis of the effects of a maximum bottom current of 4.5 fps and a maximum temperature differential for the pipe of 30°F, for extreme condi-

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[8] "Ocean Outfall Design," Hyperion Engrs., Los Angeles, Calif., 1957.

[9] "Depth of Discharge for Hyperion Sludge Outfall," by N. H. Brooks, Memorandum Report to Hyperion Engrs., Los Angeles, Calif., November, 1955.

[10] "Submarine Geology of Santa Monica Bay, California," by R. D. Terry, S. A. Keesling, and E. Uchupi, Alan Hancock Foundation, Univ. of Southern California, Los Angeles, Calif., 1956.

[11] "Report of Underwater Survey Along Proposed Hyperion Sludge Outfall," Geol. Diving Cons., Report to Hyperion Engrs., Los Angeles, Calif., January, 1956.

[12] "Report of Offshore Explorations, Proposed Sludge Line Hyperion Outfall for the City of Los Angeles," by L. L. Crandall, Report to Hyperion Engrs., Los Angeles, Calif., February, 1956.

[13] "Changes in Topography of the Beach and Foreshore in the Region of the Hyperion Sewage Treatment Plant," by R. B. Tibby, Report to Hyperion Engrs., Los Angeles, Calif., December, 1955.

tions, shows that sufficient lateral restraint will be provided by the friction of the foundation material without addition of special anchorage. As an extra precaution however, chain and block anchors were provided at intervals of 500 ft in water depths less than 150 ft.

10. Provision for mechanical cleaning of the pipe is required.

CONSTRUCTION

The 7-mile sludge outfall (Fig. 1) was constructed by a joint venture of the Healy-Tibbetts Construction Co. of San Francisco, the De Long Corp. of New York, and the Submarine Construction Co. of Port Lavaca, Tex. D. L. Narver, Jr., and E. H. Graham, Jr., have previously described[14] the construction program, that was unique in that the 7 miles of pipe were pulled as one continuous string in 7 1/2 days.

This involved making up 1,200-ft sections on a construction pier and pulling by means of a 500-ton winch located on a barge anchored seaward from the outfall terminus. During the pulling operation, the minimum radius of the vertical curve on the pipe as it left the ways was 3,000 ft and negative buoyancy was adjusted by having the line filled with air. The near-shore section of the pipe was buried to a depth of from 20 ft at the shoreline to zero at 5,900 ft offshore by means of a patented jetting device. The cost of the outfall, including engineering, was $2,719,706.

OCEANOGRAPHY

The oceanographic background, against which the operation of the outfall must be viewed, includes the water currents, temperatures, and marine life regime.

The currents in Santa Monica Bay may be classified as wind-driven, residual (related to the general circulation along the California coast), tidal, and upwelling (in which waters are brought up from depths of, 300 ft or so, towards the surface along the coast). Only the wind-driven and residual currents are of sufficient magnitude to be of concern. While surface currents in excess of two knots (200 fpm) have been observed, 90% of the currents are less than 0.5 knot, and the average is about 0.3 knot. The velocities decrease with depth and at 300-ft depths are about 0.1 knot. The circulation pattern of the Bay is such that surface waters, that are influenced by the afternoon sea breeze, typically move shoreward in a counter-clockwise gyral from the outfall area and leave the Bay along its northern shore. Currents at depth, in addition to being slower, are generally parallel to the coastline.

Tidal currents, while low, may nevertheless cause local mixing in areas of high relief such as the edge of the shelf and the head of the submarine canyon at which the outfall is located. Analysis of subsurface temperature observations made since 1955 indicates that this mixing forms large dome-shaped masses of cold water that move slowly along the bottom, under the influence of the prevailing current.

Intermittent upwelling in the bay is caused by offshore movement of coastal surface waters under the influence of wind and is the subsequent replacement

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[14] "Two Long Ocean Outfalls Constructed," by D. L. Narver, Jr. and E. H. Graham, Jr., Civil Engineering, Vol. 28, No. 1, 1958, p. 6.

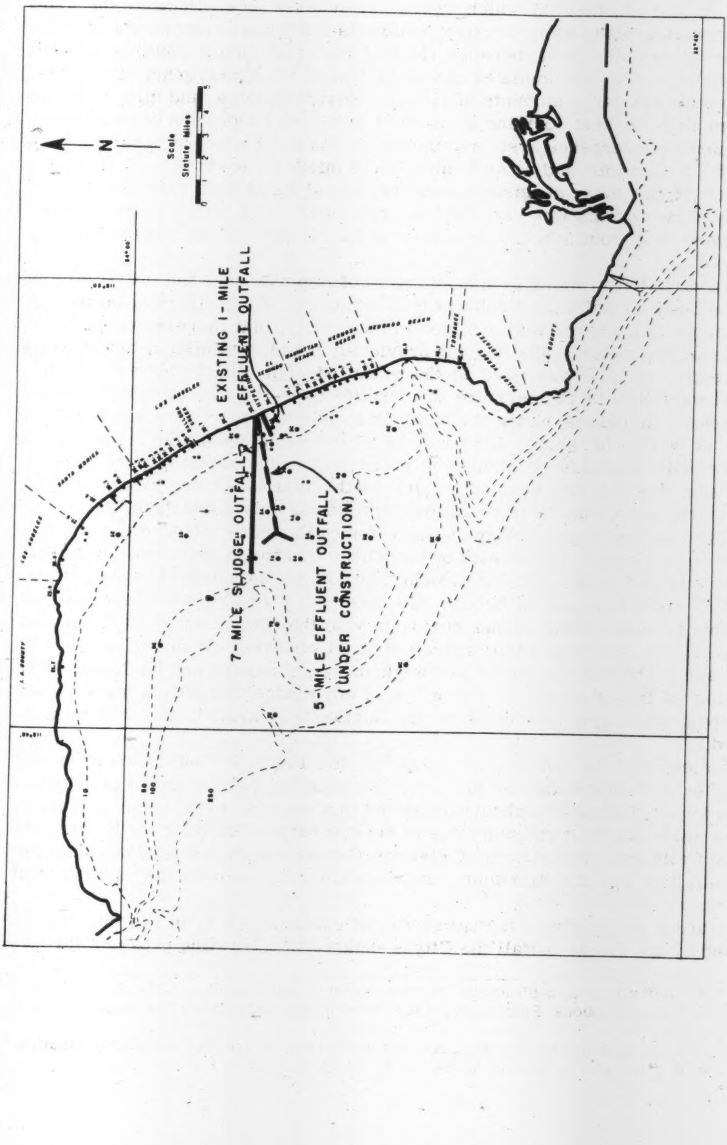
N
Scale
Statute Miles
0
1
2
3
4
5
EXISTING 1-MILE
EFFLUENT OUTFALL
7-MILE SLUDGE OUTFALL
5-MILE EFFLUENT OUTFALL
(UNDER CONSTRUCTION)
SANTA MONICA
LOS ANGELES
EL SEGUNDO
MANHATTAN BEACH
HERMOSA BEACH
REDONDO BEACH
TORRANCE
L.A. COUNTY
10
50
100
250

FIG. 1.—HYPERION OUTFALLS AND WATER SAMPLING STATION LOCATIONS

of these waters by water from depth. Velocities are very low, of the order of 30 ft per month, and may be neglected.

A characteristic of the hydrography of Santa Monica Bay is the persistence of convergence systems throughout most of the year. A convergence is the place on the surface at which two water masses meet. Because the water level remains essentially constant, there is a downward movement of water at the convergence, and, because floating material cannot continue to move with the water, it accumulates along the line of the convergence. During the warmer months large amounts of spores, oil from diatoms, and miscellaneous flotsam may be seen forming scum lines and sleeks along the convergences. Continuous convergence systems marked in this way have been observed to be over 50 ft in width and some 5 miles to 10 miles in length. Fig. 2 shows the location of the most persistent convergence systems that were observed on weekly surveys from October, 1957 to September, 1958. Other systems exist, but were not routinely observed because of the course pattern that was followed.

The variation in water temperatures is of importance in the initial dilution and the depth at which the discharge field stratifies. This follows from the fact that temperature is the major factor in the density, that increases with depth, of normal sea water in the Bay. As previously noted, a minimum temperature differential of 5°F was used in the outfall design. Fig. 3 shows the typical annual variation in temperature differential over the sludge outfall. As the differential increases during the summer months due to surface warming, the depth at which the sludge field will be at the same density as the receiving waters will increase. It should be noted that differentials of less than 5°F represent the development of a thick isothermal layer due to wave mixing during winter storms. This condition persists for only a few days.

While studies of the biology of Santa Monica Bay have included the plankton in the upper layers, any effects of the sludge discharge are normally limited to the attached, creeping, or crawling bottom organisms and to the bottom-dwelling fish. Changes in bottom life take place slowly, and it has not been possible to observe any change during the year that the sludge outfall has been in operation. We have some indication of the changes that might be expected from any portion of the sludge that might move shoreward and be deposited on bottoms of less than 320 ft. The effect of any sludge that moves seaward and is deposited at greater depths in the submarine canyon cannot now be estimated.

Studies[15] of the bottom organisms by the Hancock Foundation show that the effects of suspended solids from the existing 1-mile outfall may be observed up to distances of about 6 miles and that the zone between about 2 miles and 4 miles shows enrichment due to the discharge. Trawling for bottom fish by the California Department of Fish and Game showed no significant effect of the outfall except that maximum catches were obtained over the same zone of enrichment.[16]

In order to provide a factual record of changes in bottom life that may be caused by the sludge outfall the City is engaged in a trawling program through

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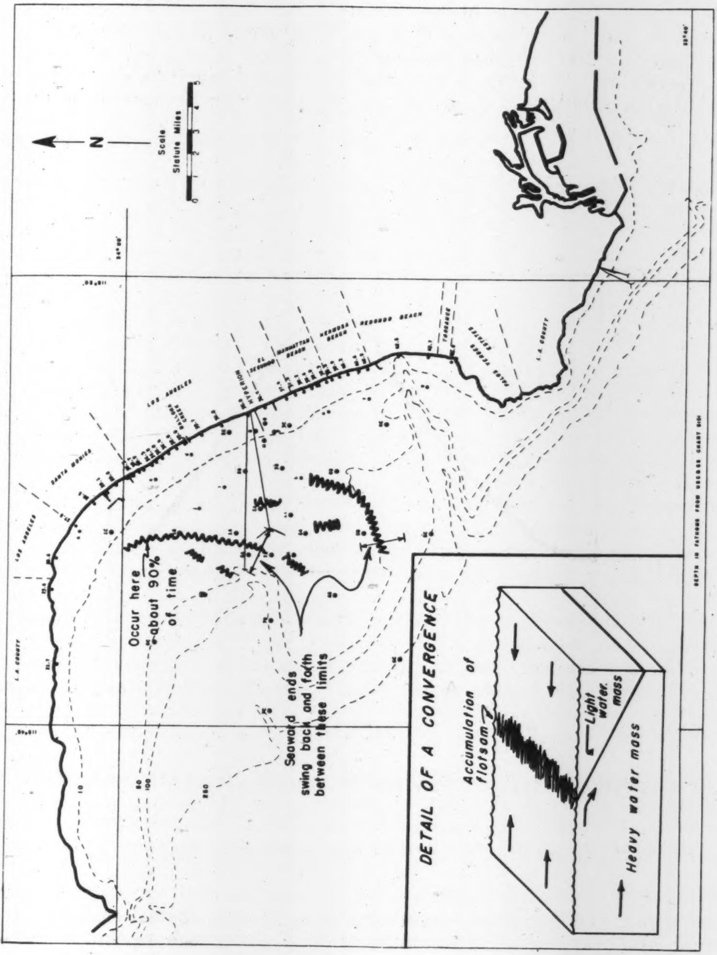
[15] "Contributions to a Biological Survey of Santa Monica Bay, California," by Olga Hartman, Alan Hancock Foundation, Univ. of Southern California, Los Angeles, Calif., 1956.

[16] "Sewage in Santa Monica Bay, A Critical Review of the Oceanographic Studies," by P. L. Horrer, et al., Marine Advisors, La Jolla, Calif.


N
Scale
Statute Miles
0
1
2
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5
LOS ANGELES
SANTA MONICA
L.A. COUNTY
HYPERION
EL SEGUNDO
MANHATTAN BEACH
HERMOSA BEACH
REDONDO BEACH
TORRANCE
PALOS VERDES ESTATES
10
50
100
250
Occur here about 90% of time
Seaward ends swing back and forth between these limits
DETAIL OF A CONVERGENCE
Accumulation of flotsam
Light water mass
Heavy water mass

FIG. 2.—CONVERGENCES, OCTOBER 1957 TO SEPTEMBER 1958

which both bottom dwelling fish and such invertebrates as crabs and shellfish are collected. Trawls are made seasonally in order to evaluate natural variations and will be continued so that the actual effect of the outfall can be determined.

The estimated rate of sedimentation of sludge in the area around the outfall was developed[17] by Brooks. At the ultimate design flow, solids deposition is estimated to vary from about 0.50 lb per sq ft per yr at 1,000 ft to 0.17 lb per sq ft per yr at 10,000 ft from the point of discharge. Bottom cores taken in the immediate vicinity of the outfall after 3 months of operation showed only a thin film of identifiable sludge solids. The problems of sampling such a film in waters of 300-ft depths together with the bacteriological changes that affect the film over a period of time make evaluation of sludge deposition itself of doubtful importance. A much more realistic approach is to collect the bottom

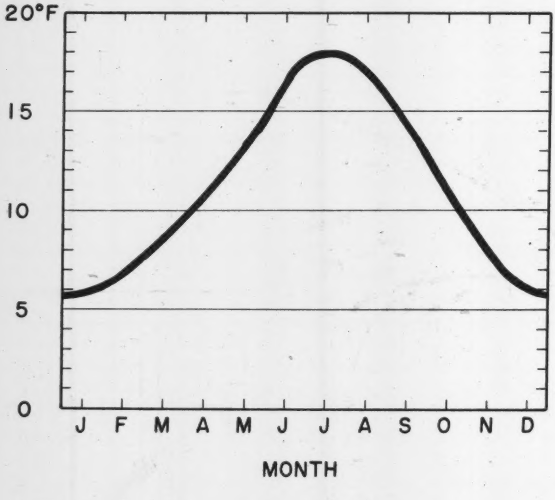

FIG. 3.—TYPICAL ANNUAL VARIATION OF TEMPERATURE DIFFERENTIAL

fish and other organisms that may be affected by the sludge. This is being accomplished by the trawling program.

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[17] "Predictions of Sedimentation and Dilution of Digested Sludge in Santa Monica Bay," by N. H. Brooks, Report to Hyperion Engrs., Los Angeles, Calif., August, 1956.

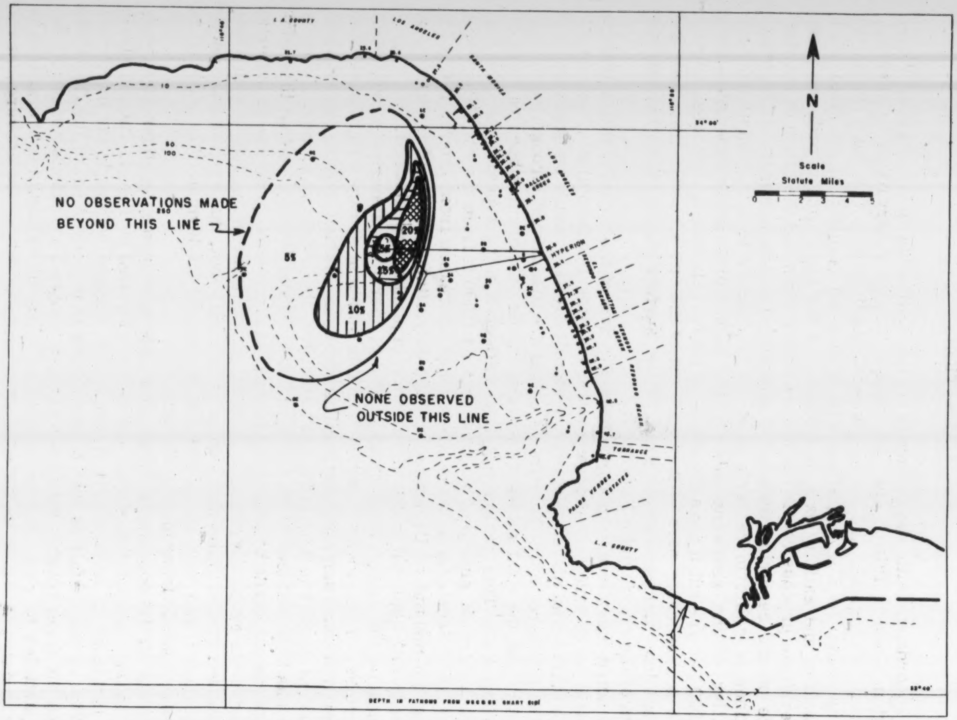
OPERATIONS

In accordance with requirements of the California State Regional Water Pollution Control Board, the city makes weekly observations of physical, chemical, biological, and bacteriological conditions at the water sampling stations shown on Fig. 1. Certain information not required by the state, such as plankton and salinity data, is collected at the same time in order to more fully evaluate process control in the treatment plant and the effect of the discharge on the receiving waters. In addition, the city has made daily determinations of coliform bacteria at surf sampling stations located along some 15 miles of the shoreline since 1946. Such a program is expensive—over $100,000 per yr—but it pays dividends in public relations and process control and is a reasonable part of the total cost of the city's sewage treatment and disposal into marine waters. Some of the surface observations indicate the effects of the sludge outfall operation.

As noted elsewhere,[5] approximately one-half of the digested solids leaving the plant were formerly discharged through the 1-mile outfall. Because the sludge outfall was placed in operation with all digested solids being discharged 7 miles offshore, only the effluent solids are discharged through the 1-mile outfall. Thus, the total solids discharged 1 mile offshore has been reduced by about one-half. This change has been reflected in the appearance of the boil and the water at smapling stations located 1,000 ft from the outfall at which the average Secchi disk transparency has increased from about 6 ft to 11 ft during the months of December through July, although some of the increased transparency is probably due to variation in plankton populations.

Because the sludge outfall was placed in operation, a marked reduction in the number of occasions on which floatable solids of sewage origin have been observed at beach sampling stations has also been obtained. During the 4-month periods of April 8 to August 8 in 1957 and 1958, the incidence of such observations has been reduced from 1.04% to 0.07%. It should be noted that these observations are made by trained personnel who report on presence of material that consists primarily of small, partially decomposed rubber flakes with some particulate grease balls about the size of small shot, that are, on rare occasions, associated with small flocs of sludge. To the layman, this material is indistinguishable from marine flotsam.

During initial operations, floatable solids have also been observed in surface waters around the sludge outfall as indicated on Fig. 4. The relatively low incidence of such observations in the surface waters in the immediate vicinity of the outfall is due to the downstream movement of the floatables during the time, roughly estimated at from 2 hr to 4 hr, that is necessary for the particles to rise. The accumulation of the floatables in convergences is quite consistent. Thus, we have evidence of the ability of the convergences to accumulate and retain material for sufficient periods of time to allow further alteration with the result that they cannot be found at surf stations downcurrent from the convergences in and around Santa Monica. It must be emphasized that recognition of the small amount of floatables of sewage origin that are included within large amounts of marine debris is possible only by trained personnel.


N
Scale
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0 1 2 3 4 5
34°00'
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L.A. COUNTY
LOS ANGELES
SANTA MONICA
BALLONA CREEK
LOS ANGELES
HYPERION
EL SEGUNDO
MANHATTAN BEACH
HERMOSA BEACH
REDONDO BEACH
TORRANCE
PALOS VERDES ESTATES
L.A. COUNTY
10
50
100
NO OBSERVATIONS MADE BEYOND THIS LINE
NONE OBSERVED OUTSIDE THIS LINE
5%
10%
15%
20%
DEPTH IN FATHOMS FROM USC&GS CHART

FIG. 4.—SLUDGE OUTFALL OPERATION (OCTOBER 1957 TO SEPTEMBER 1958) OCCURRANCE OF FLOATABLES

Although the amount of floatables discharged from the treatment plant is extremely small, their presence in any amount is both undesirable from esthetic considerations and prohibited by the requirements of the Water Pollution Control Board that state, "There shall be no oily sleek or no floating or suspended solids recognizable by eye as of sewage origin. . . ." Such requirements, that necessarily involve a 100% efficiency in removal of floatables by the treatment plant, came close to implying the golden age in sewage treatment.

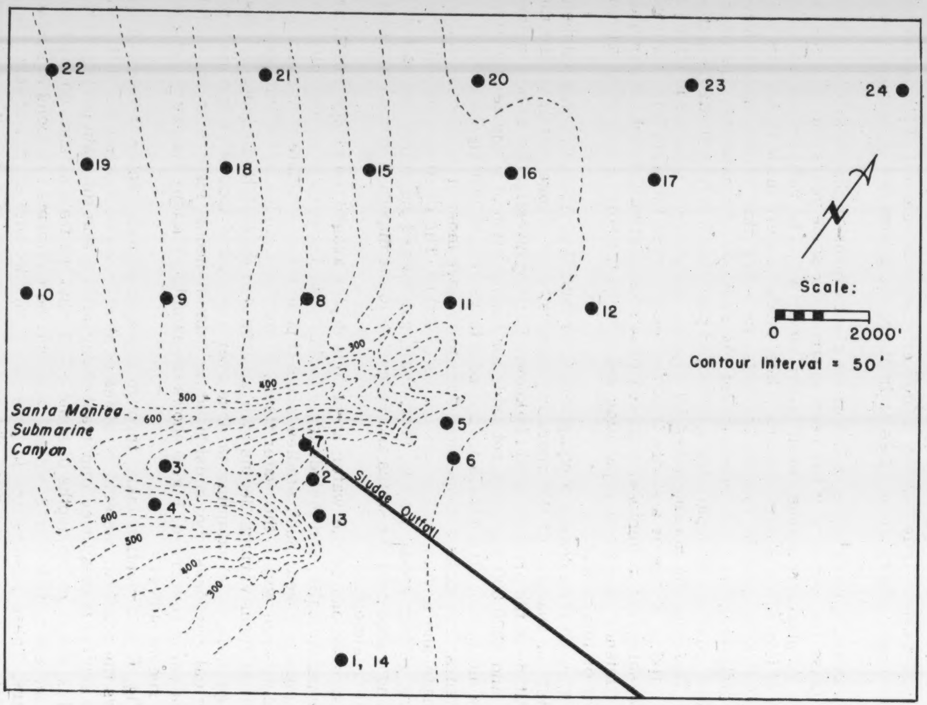
It was recognized in the design of the plant expansion that floatables might give cause for concern. It has been apparent for some years that changes or upsets in plant processes, even minor ones, would be reflected in the incidence of floatables at surf stations. Thus, studies[8] were made by the city and Hyperion Engineers of various methods, including fine screening, mechanical disintegration, and impingement for removal of floatables. The basic problem consists of separating gas buoyed particles and true floatable material. This may be accomplished by violent agitation or washing in either a flash mixing device or a sludge impingement process (wherein jets of sludge and washing water impinge) followed by a small skimming tank. In general, the efficiency of such a separation depends on the horsepower input (either mixing or impingement).

Action on final design of equipment for removal of floatables was withheld pending an analysis of the operating results of the sludge outfall so that the magnitude of the problem would be known. Further investigation is now underway, and $200,000 has been set aside for construction of adequate facilities. In the meantime, all sludge is processed in tanks originally designed as elutriation tanks. The tanks are operated so as to remove most of the flow as underdrainage and a very minor amount as overflow. The floating material (containing rubber goods, grease, hair, chaff, gas buoyed particles, and so forth) is hand raked over the weirs and re-cycled to the digestion system for additional treatment.

Examinations of the water at various depths around the sludge outfall are made periodically, and include tests for bacteria, temperature, chlorinity, and ammonia-nitrogen. Such studies make it possible to evaluate the behavior of the discharge within the water itself. From 100 samples to 150 samples are required from various depths over an area that essentially includes the discharge field. The sampling is done over a period of about 8 hr and may, for practical purposes, be considered as simultaneous insofar as the outfall operation is concerned. Fig. 5 shows the sampling stations that were occupied on a recent survey.

Fig. 6 is a section taken through the approximate center of the field and shows the coliform bacteria densities as Most Probable Number per milliliter. This section makes three important observations: that the main portion of the field is restricted to depths of some 200 ft, that the field extends downstream for an estimated 1 1/2 miles, and that no coliforms are found in surface waters. These results are in general conformity with earlier observations.

Water temperatures were determined by means of a bathythermograph. This provides a continuous trace of temperature versus depth on a smoked glass slide, that is then read by means of a calibrated grid. Fig. 7 shows temperatures along the same section for which the coliform data were plotted. It is seen that the upper surface of the coliform field lies in water of from 56°F to 57°F, and that the temperature differential between the outfall depth


22
21
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10
9
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Scale:
0
2000'
Contour Interval = 50'
300
400
500
600
Santa Montea
Submarine
Canyon
5
7
6
3
2
4
13
Sludge Outfall
600
500
400
300
1, 14

FIG. 5.—SLUDGE OUTFALL OPERATION, SAMPLING STATION CHART

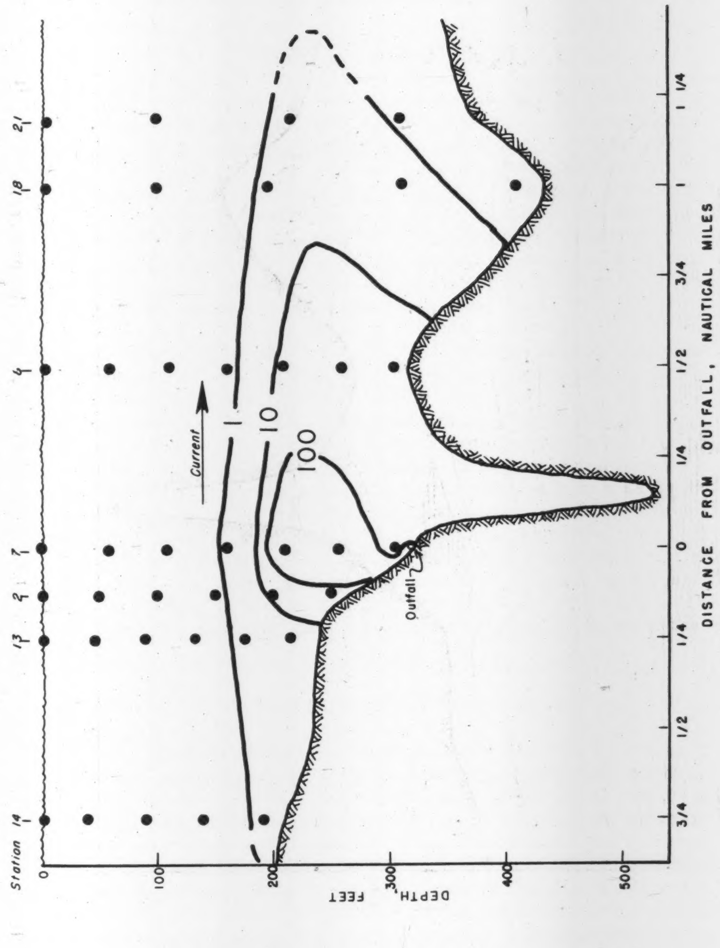

FIG. 6.—SLUDGE OUTFALL OPERATION, COLIFORM BACTERIA

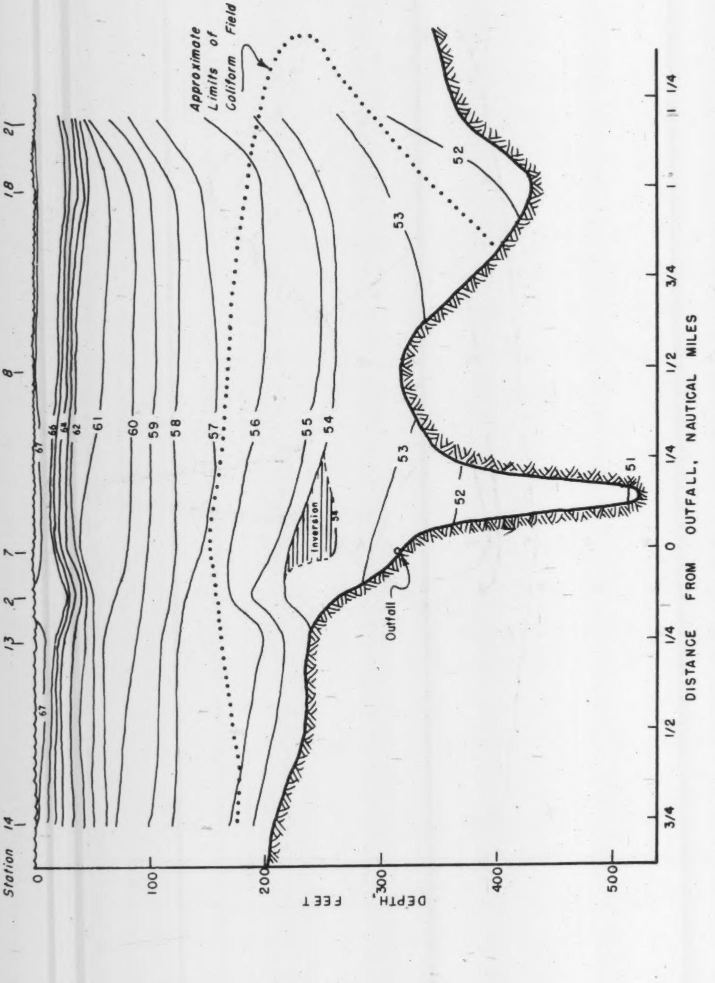

FIG. 7.—SLUDGE OUTFALL OPERATION, WATER TEMPERATURE

and the limit of the field is about 4°F. This temperature differential is encouraging. It is in good agreement with theory in which a 5° differential was assumed, and it is less than the typical winter-time minimum.

The chlorinity and ammonia-nitrogen data are in general agreement with the preceding data, although normal variations of these components in the receiving waters do not permit detailed comparisons.

CATHODIC PROTECTION

The line is provided with an impressed-current type of cathodic protection system in addition to the interior and exterior coatings in order to provide maximum protection against corrosion. Power is supplied from the Hyperion plant via a 50 amp rectifier. The anodes are made of durichlor, a material that was developed to give long life in sea water service. The anode bed is located in the surf zone astride a conveniently placed storm drain outlet approximately 550 ft from the point at which the sludge outfall enters the sea.

A unique feature of the cathodic protection system is the series of test leads that are attached to the pipe at various points in the ocean and brought to test stations ashore via a submarine cable. Rusty iron and zinc anodes used as reference, are buried in the ground adjacent to the pipe and electrical leads are brought up to the test stations. With this system it is possible to quickly determine the protection given the pipe at various points by means of electrical potential measurements.

Approximately 0.180 amp at 2.50 v have been applied to the system in order to provide protection. This very low current requirement is due to exceptionally excellent condition of the pipe coatings. It is anticipated that the current requirements will increase somewhat in the ensuing years as the coatings deteriorate. There is adequate capacity, of course, in the 50 amp rectifier. A potential of between 1.05 v and 1.10 v is maintained between the pipe and a copper sulphate reference cell. Current readings are taken daily and potential measurements are made weekly. Ajustments are made as necessary to maintain the desired potential. The consulting engineer who designed the system reviews the operating data and makes occasional special tests as required.

PIPELINE PERFORMANCE

As previously noted, provision was made for cleaning the line at such times as experience would indicate. With one year's operation completed, analysis of the sludge pumping requirements indicates that the Darcy f of the line has increased by 23%. While the line obviously still has adequate capacity, plans are now underway for mechanical cleaning of the line. The increase in the friction factor may also be expressed as a reduction in the diameter of the pipe amounting to about 0.4 in. It can be speculated that this is due to the cooling of the sludge discharge by about 10°F during the flow time through the pipe and the resulting deposition of residual grease, that may remain even after complete digestion, on the pipe wall. This effect is, of course, even more pronounced with raw sludge as was found in Oakland where, during the winter months, grease deposition in sludge lines has been an operating problem.[18]

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[18] "Raw Sludge Pumping Experiences at East Bay Municipal Utilities District Plant," by Elmer Ross, presented at the April, 1958 CSIWA Conference.

However, the records obtained from cleaning sludge lines within the city and elsewhere indicate that this is not considered to be of major significance in the successful operation of the sludge line.

The operating cost of the new sludge disposal method is expected to be about $1.15 per ton as formerly represented by the cost of elutriation only. The power cost is less than 10% of that required for the filter and dryer operation.

SUMMARY AND CONCLUSIONS

One year's experience with a 7-mile, 21-in. dia outfall discharging about 5 mgd of digested sludge and plant effluent into 320 ft of water in Santa Monica Bay has shown that the operation has fulfilled the expectations of the designers. The sludge field rises until it is in equilibrium with the surrounding water and then flows with the prevailing current. The water at the equilibrium level is about 4°F warmer than that on the bottom and it is indicated that, except under extreme conditions, the sludge field will be well submerged. Floatables, that may be recognized by trained personnel as being of sewage origin, have been observed in the Bay waters, and additional works for removal of this material are in an advanced planning stage.

ACKNOWLEDGMENTS

The authors are indebted to W. A. Schneider, Director, Bureau of Sanitation; G. A. Parkes, W. F. Garber, M. ASCE, J. R. Stanton, and M. E. Nelson, M. ASCE, Engineer-Superintendent, Laboratory Director, Chemist, and Biologist, respectively, Hyperion Treatment Plant; D. L. Narver, Jr., F. ASCE, Project Manager, and E. H. Graham, M. ASCE, D. R. Miller, M. ASCE, and C. H. Lawrance, M. ASCE, of Hyperion Engineers whose efforts throughout the design, construction, and operation of the sludge outfall have made this study possible.

DISCUSSION

CHARLES H. LAWRANCE,[19] M. ASCE, and DAVID R. MILLER,[20] M. ASCE.—This paper has been most interesting to the writers, who were among the designers of the facility discussed. It is particularly gratifying to note the significant improvement observed in the incidence of floatable solids of sewage origin at beach sampling stations and the general improvement in the conditions of the receiving waters after the outfall was put into operation. Because of the extensive monitoring program of the city, covering both shore and offshore stations in Santa Monica Bay, additional valuable data and conclusions may be expected in years to come concerning submarine disposal of sludge and of treatment plant effluent.

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[19] San. Engr., Koebig and Koebig, Cons. Engrs., Los Angeles, Calif.; formerly San. Engr., Hyperion Engrs., Los Angeles, Calif.

[20] Chf. Transit Engr., Daniel, Mann, Johnson and Mendenhall, Archts. and Engrs., Los Angeles, Calif.; formerly Proj. Engr., Hyperion Engrs., Los Angeles, Calif.

The decision to replace the filter and drier facilities at Hyperion with ocean disposal was one of the most controversial aspects of the entire Hyperion Expansion Program. Many persons argued that it was an inconceivable waste of a valuable natural resource; others that the discharge of this material into a bay having no strong, dominant offshore currents would result in an intolerable pollution of the bay. Still others contended that a separate sludge facility was a sheer waste of money and that the sludge should be discharged with the effluent through the ocean outfall for effluent disposal. The authors have presented the arguments refuting these contentions. The writers desire to elaborate upon some of the points at this time.

The fertilizer project to which the authors alluded was given every chance to succeed, for a number of years, in the face of obstacles that arose in solids-removal and in processing and marketing the dried sludge. Since sewage sludge is an unavoidable by-product of the larger overall process of sewage treatment, the cost of its disposal is rightfully apportioned within the total cost of sewage treatment. If it is possible to utilize it directly as fertilizer on land - as opposed to fertilizer for marine life - and if revenues are obtainable to defray the costs of its disposal, so much the better. However, sewage sludges, by their very nature, are not in a position to compete openly with inorganic commercial fertilizers on the basis of fertilization potential.

Hyperion fertilizer contains approximately 2.5% nitrogen, 3.5% phosphoric acid, and no potash. Digestion operations affect the nitrogen content, as described by W. F. Garber, M. ASCE.[21] A typical commercial inorganic fertilizer, 8-8-4, sells in bulk in the Los Angeles area for about $47 per ton, representing a price of $588 per ton each of nitrogen and of phosphoric acid. Another commonly used fertilizer, 16-20-0, sells for about $80 per ton, corresponding to $500 per ton of nitrogen and $400 per ton of phorphoric acid. Based upon these fertilizer indices, a comparable sales price to the bulk consumer would be about $15 per ton for the Hyperion dried sludge. The disparity between this figure and the $4 per ton for which the sludge is actually sold to a fertilizer firm for subsequent marketing apparently represents handling, transportation, and other charges on the part of this firm and reflects some of the difficulities in marketing the dried sludge. Potential markets have included crop-growers within the Los Angeles Basin and environs, including even those located in the Imperial Valley some 200 miles southeast of Los Angeles. However, for a variety of reasons ultimately determined by the economics confronting the grower applying the fertilizer, inorganic fertilizers have generally been found preferable despite the soil-conditioning advantages of the dried sludge, and this market has never fulfilled expectations. Potential retail markets for sale of sewage sludges to householders have encountered sales resistance. Problems in the production of the sludge, particularly in the elutriation, conditioning, and vacuum filtration processes at Hyperion have been presented comprehensively in the literature by R. D. Bargman, F. ASCE, W. F. Garber, M. ASCE, and J. Nagano.[22]

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[21] "Plant-Scale Studies of Thermophilic Digestion at Los Angeles," by W. F. Garber, Sewage and Industrial Wastes, Vol. 26, No. 10, October, 1954, p. 1202.

[22] "Sludge Filtration and Use of Synthetic Organic Coagulants at Hyperion," by R. D. Bargman, W. F. Garber, and J. Nagano, Sewage and Industrial Wastes, Vol. 30, No. 9, September, 1958, p. 1079.

The experiences of the city of San Diego, Calif. as described by R. C. Merz, F. ASCE[23] have shown that it is more economical for San Diego to dispose of digested sludge from its primary treatment plant onto waste land as a reclamation project for agricultural purposes than to operate its existing sludge conditioning and drying facilities. The processed sludge was formerly being put directly to beneficial use and the city's decision to continue such use, in the modified manner selected, represents an encouraging reversal of the trend towards disposal of the sludge by unreclaimable means and is a demonstration of the fact that under the right circumstances, this natural resource can be exploited profitably. In the case of San Diego, the City already owned available waste lands and was able to transport the liquid-digested sludge to them by tank truck for subsequent spreading.

Reportedly the cost of fertilizer production had been $39.15 per ton of dry solids handled in operations similar to those for which the authors have cited their costs of $14.22 at Hyperion. At San Diego, the changeover to land disposal from the original method of fertilizer production for contract sale to a fertilizer firm has resulted in a savings of $16.12 per ton of dry solids, leaving a cost of $23.03 per ton for land disposal. The San Diego plant handles approximately 60 mgd as compared to Hyperion's 270 mgd; this flow difference is undoubtedly a major factor entering into the differential in unit treatment cost between the two plants.

The overall environmental picture, embracing the aspects of water, air, and food supply is gradually being drawn more and more into focus in the eyes of the engineer as populations swell and by their growth point to a possible eventual saturation point in certain areas. Mark Hollis,[24] F. ASCE, among others, has sounded a warning regarding the outlook for water pollution in the United States. Others may be expected to follow. The author's own observation that man sometimes exchanges one pollution problem for another is poignantly true in the case of the Los Angeles Basin. In this area not only has the Hyperion Treatment Plant been squeezed between air pollution limitations from sludge-drier stack emissions on one hand the water pollution limitations from sludge elutriate discharge on the other hand, but refuse incinerators for individual dwellings are now extinct and municipal incinerators are gradually giving way to land disposal which may in itself present some potential problems of ground water pollution.

It is difficult to compare the operating costs of the two methods of sludge disposal, fertilizer production and submarine discharge, because of the many factors involved. Some of these are tangible, such as power consumption and availability, and some tend to be intangible, such as effects upon other plant processes and effects upon the receiving waters. A crude economic comparison for the best year in solids removal with 92 tons per day or 57.5% of the total digested sludge production being processed and the remainder being lost to sea via the one-mile ocean outfall would show a cost of the processing at $14.22 per ton equal to $1,318 per day. This would compare with 160 tons per day at $1.15 per ton or $185 per day for sludge outfall discharge. A strong advantage in favor of the sludge outfall is shown. Addition of the fixed charges to the comparison would probably add more to the advantage of the sludge outfall.

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[23] "Utilization of Liquid Sludge," by R. C. Merz, Water and Sewage Works, Vol. 106, No. 11, November, 1959, p. 489.

[24] Engineering News-Record, Vol. 163, No. 17, October 22, 1959, p. 24.

The cost of ocean disposal will ultimately be increased because of the cost of equipment which will be required to remove floatables, but this will not materially change the sludge outfall's economic advantage. These floatables, originating with the sludge outfall discharge and recognizable only by trained personnel as being of sewage origin, have already been reduced in their incidence at beach sampling from 1.04% to 0.07%. In the face of these excellent operational results, the fact that the city of Los Angeles still plans to go ahead and spend an additional $200,000 to remove even this small amount of residual floatables is a testimony to the desire of the city to do its utmost to maintain the best possible conditions in the waters of Santa Monica Bay.

Present pilot plant work on removal of floatables from digested sludge employs a compartmented rectangular tank in which the digested primary sludge is diluted with plant effluent and then is successively subjected to rapid mixing, slow stirring, and sedimentation to promote separation of the fractions which appear as floating material on the ocean's surface from time to time. The most conspicuous material is rubber in the form of thin flecks and occasionally fragments of rings originating with rubber goods. Cellophane particles are quite commonplace, but these exhibit a tendency to settle as well as to float both in sea water and in fresh water. Large volumes of hair occur in the sludge along with other constituents such as humus, grease particles, undigested vegetable matter, and occasional cigarette filter tips. Following rapid and slow mixing in the tank, the worst offender, rubber goods, is largely floated off as effluent scum in the sedimentation compartments along with considerable hair and grease. The sludge humus mostly settles to the bottom, but the cellophane, hair and vegetable matter may be found in both the tank scum and bottom sludge and even, to a lesser extent, in the intermediate depths of the tank. Work is now proceeding to better establish the relationships between removal of floatables and such parameters as power application and overflow rate, prior to development of final design.

The design criteria for the sludge outfall were established only after exhaustive studies had been made upon the settling characteristics of the digested sludge and the oceanographic conditions of Santa Monica Bay. These latter conditions, as summarized by the authors, are typified by onshore counter-clockwise surface currents and currents at depth which are generally parallel to shore and are relatively slow. The existence of convergences offshore is noted with the observation that these seem to act as barriers to prevent floating material from reaching the beaches. While occasional swimming and other aquatic sports activity extend for an appreciable distance offshore, by far the majority of use normally occurs within 1,000 ft of shore. The convergences have a beneficial effect in discouraging floating material from entering this nearshore zone.

The oceanographic studies measuring temperature variations in the area of proposed sludge discharge and determining the subsurface current pattern were made for the purpose of establishing the required length of the outfall and the depth of discharge necessary. The intention was always to discharge the sludge at sufficient depth to provide mixing with cold bottom water so that the resulting density of the sludge-sea water mixture would be greater than the density of the overlying warmer water and the sludge field would be kept completely below the surface. This prevents any possible unsightliness at the surface and keeps the mixture away from the stronger surface currents. Figs. 6 and 7 illustrate how this has occurred under actual operation and how the sewage field is generally kept below 150 ft in depth. These figures illustrate conditions under typical summer temperature variations. It would be interesting to see what happens in the winter season when the temperature at the point of discharge is 5°F cooler than the surface water.

Figs. 6 and 7 are also interesting from the standpoint of suggesting the dispersion of the pollutants upon submarine injection. The temperature contours of Fig. 7 shows the local heating effect of the sludge outfall discharge as evidenced by the general downward depression of the contours in the vicinity of the outlet and coliform field. Immediately above the outlet there is an upward bulge of temperature contours, apparently reflecting an induced upward movement of the cold bottom water next to the rising jet of the discharge. Within this bulge, the "inversion" or isothermal lens at 54°F. would seem to represent the rising mixture of outfall discharge and surrounding sea water. The sampling traverse was taken at an angle of 45° from the direction of the sludge discharge, following the direction of the subsurface current. The rising jet may not have been fully intersected by the profile at all points, but the profiles drawn appear quite representative.

The complete submergence of the coliform field deserves some comment. The physical dilution of the submerged "sludge field," particularly that area directly over the outlet, is very likely considerably less than that suggested by the coliform density contours in relation to the coliform density of the injected flow. This sludge and dilutent mix prior to discharge probably had a coliform concentration of slightly above 100,000 coliforms per ml, based upon the usual values experienced at the Hyperion plant. Therefore, the "apparent" dilution is up to about 100,000 at the upper limit of the coliform field, 10,000 at the coliforms per ml contour, 1,000 at the 100 per ml contour, and somewhat less than 1,000 at the bottom of the isothermal lens. The actual physical dilution could be computed from salinity and temperature data and can be approximated by methods employed by N. H. Brooks, M. ASCE,[9] and by the writers.[8] Tentatively, the physical dilutions at the bottom of the isothermal lens, the 100 per ml contour, the 10 per ml contour, and the 1 per ml contour would appear to be in the order of 30, 75, 100, and 150, respectively, these values representing the rising column issuing from the outlet. At a distance horizontally away from the column the physical dilution would be appreciably greater than in the column because of lateral diffusion experienced once the column has been sufficiently slowed in its ascent by acquiring a density approaching that of the surrounding sea water. This has apparently happened at about 250-ft depth, the approximate location of the isothermal lens and the downstream "bulge" of the coliform contours. The narrower the initial "field." the more rapid will dilution take place by the action of turbulent diffusion. The single outlet discharge into the cold bottom water would seem to reach a density equilibrium with its surroundings before it established a very wide column base or field width, hence its lateral diffusion should be reasonably rapid.

Upon establishment of the field, the complex factors of sedimentation, physical dilution, and mortality come into play in the reduction of coliform densities. These have been discussed previously by C. G. Gunnerson, F. ASCE[25] and by the writers.[8] In the case of Fig. 6, the writers have insufficient information to offer anything more than generalizations regarding the nature of the coliform contours. Downstream of the outlet, the distance between contours of one magnitude (90% reduction) is about 3,000 ft, especially at the "bulge." This is believed to correspond to 3 to 5 hr time in the direction of the current. Assuming a stratification of field initially at about 230-ft depth, equivalent to a stratified sewage field on the surface of the ocean, the overall T-90 or time for 90% coliform reduction would be about 4 hr. Sedimentation of coliforms takes place rather slowly judging by the spread of the contours. This would imply that the sludge suspension itself settles fairly slowly also. Additional bottom core samples and trawls should clarify this matter. The writers are unable to explain the seeming discrepancy between expected physical dilution and the "apparent" bacterially-measured dilution of the outfall discharge in the depths lying more or less directly over the outlet. It may be that the subsurface field stratifies fairly rapidly except for a small portion of rising column which carries up to above the 200-ft depth. In any event, local coliform concentrations much higher than 100 per ml over the ourfall would be expected unless the sampling chanced to miss them.

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[25] "Sewage Disposal in Santa Monica Bay," by C. G. Gunnerson, Transactions, ASCE, Vol. 124, 1959, p. 823.

The authors state that after one year's operation, the Darcy f of the line has increased 23%. Even with the increase, the line still has adequate capacity, and the decision to clean the line mechanically was apparently a routine maintenance procedure. Since this cleaning has now taken place, it would be interesting to know how much the cleaning was able to reduce the friction value and if there is any tendency for an immediate return of the depositon of grease which the authors assume is the cause of the increased roughness. Is there any plan for the regular use of the cleaning device on a regular maintenance program?

NORMAN B. HUME,[26] F. ASCE, ROBERT D. BARGMAN,[27] F. ASCE, CHARLES G. GUNNERSON,[28] F. ASCE, AND CHARLES E. IMEL,[29] A. M. ASCE.—Lawrance and Miller have commented upon the behavior of the floatables, apparent dilutions, sedimentation processes, and the cleaning of the sludge line. Observations made during the 2-yr period following that of the original investigation provide further information on these subjects.

Studies of various methods for removal of floatables have continued and it is expected that works for their removal will be in operation within a reasonable period. In this connection, waste activated sludge was pumped to sea and the receiving waters were closely studied. Microscopical examinations were made of water samples from various depths. It was found that the activated sludge floc, and particularly sphaerotilus, was elutriated from the discharge and distributed throughout the upper 100 ft while the field itself remained at depths generally greater than 200 ft. Accordingly, digestion is necessary for waste activated sludge which is to be discharged at sea.

Additional studies designed to evaluate initial dilution in the rising column have been made. It should be noted that only conservative properties such as temperature, salinity, or dissolved tracer concentration can be used for dilution computations. Non-conservative properties such as coliform density and suspended solids concentration provide information only upon the fate of the particular constituent. It has been found from detailed studies that as the discharge column rises through a temperature differential of about 4° F which usually obtains between the 200-ft level and the bottom, a minimum dilution of 75: 1 is effected.

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[26] Asst. Dir., City Bur. of Sanitation, Los Angeles, Calif.

[27] Asst. Chf. Engr., City Sewage Treatment Div., Los Angeles, Calif.; formerly, Asst. Engr.-Supt., Hyperion Treatment Plant, Los Angeles, Calif.

[28] Civ. Engr., Bur. of Sanitation, Los Angeles, Calif.

[29] San. Engrg. Assoc., City Pub. Works Dept., Venice, Calif.; formerly, San. Engrg. Assoc., Hyperion Treatment Plant, Los Angeles, Calif.

Bottom samples taken over the 3-yr period of operation have shown that the maximum accumulation of sludge on the bottom has been held to less than 1 ft. This is the result of stabilization of the material by bottom organisms, intermittent winnowing by bottom currents, and periodic slumping of the deposits into the submarine canyon.[30]

One detailed survey of the receiving waters was made in order to characterize the deposition pattern of the sludge. Quantitative microscopical analyses involving staining of the entrained sludge with methylene blue were made of water samples from various depths throughout the receiving waters. The deposition rate of the sludge was estimated from the concentrations in the water. This indicated rate reached a maximum at a distance of about 500 ft from the outfall and became negligible at about ½ mile. These data are generalized in Fig. 8, which is based upon a bulk density of 10 pcf for the deposit.

Because of several variables in the observations and analytical techniques, the absolute values of the deposition rates are somewhat uncertain, although the relative values are probably good.

When viewed by underwater television, the superficial bottom deposits were found to vary between 1 in. and 6 in. in thickness and were in almost continuous motion from the swash of bottom waters.

Considerable physical and biological activity has been noted in the deposition area. Some 30% to 40% of bottom deposits were found to consist of worm castings. The bottom deposits are not a firm material but are compacted to a bulk density of about 10 pcf. There has been no significant variation in the variety of species of fish and invertebrates in the area although some of the detritus feeders have shown large increases in population. Winnowing of the deposits is quite evident from the bottom sampling program which shows a periodic build up and removal of both total deposits and percent fines.

Other effects of bottom currents were found during a recent inspection of the pipe line in depths of from 60 ft to 145 ft. As noted by the authors, a series of anchors spaced at 500-ft intervals were installed to prevent lateral movement. Each anchor unit consists of two 4,500 lb concrete blocks connected by a 6-ft chain that lies across the pipe. A scoured area was found around each of the anchor blocks at all depths between 100 ft and 145 ft, the limit of the inspection. The scour is about 1 ft deep and extends about 10 ft in all directions from the anchors. The pipe rests on the bottom in these areas. Normally it is bedded to just below the spring line. Along the pipe between the blocks there is sand deposition of from 2 in. to 8 in.

The sludge outfall cleaning, which was anticipated after 1 yr of operation, has been accomplished. A two-pass operation was used in the cleaning. A "prover" consisting essentially of a pair of 20-in. diam plywood circles (Fig. 9) was first sent through the line to insure that there would be no restriction that would prevent the passage of the articulated "pig" (Figs. 10 and 11), which included brush and scraper elements. Both the prover and pig passed through the line in about 3½ hr, which is essentially the flow time of the diluted sludge.

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[30] "Characteristics and Effects of Hyperion Effluent in Santa Monica Bay, California," by N. B. Hume, C. G. Gunnerson, and C. E. Imel, presented at the 1960 Meeting, of the Pacific Sect., of the Amer. Geophysical Union, at the Univ. of Southern California, at Los Angeles, Calif.

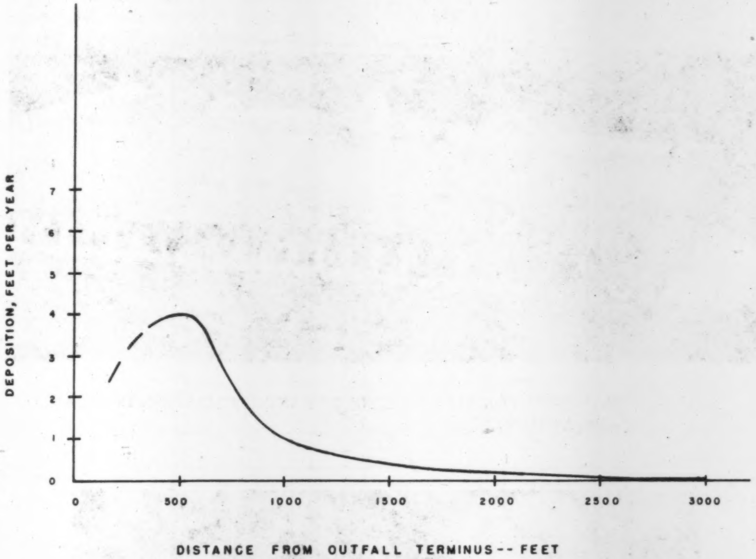

FIG. 8.—ESTIMATED RATES OF SLUDGE DEPOSITION AS DETERMINED FROM SUSPENDED SOLIDS CONCENTRATION IN WATER COLUMN

FIG. 9.—"PROVER" USED FOR SLUDGE LINE CLEANING. THE CIRCLES HAVE A DIAMETER OF ABOUT 20 IN.

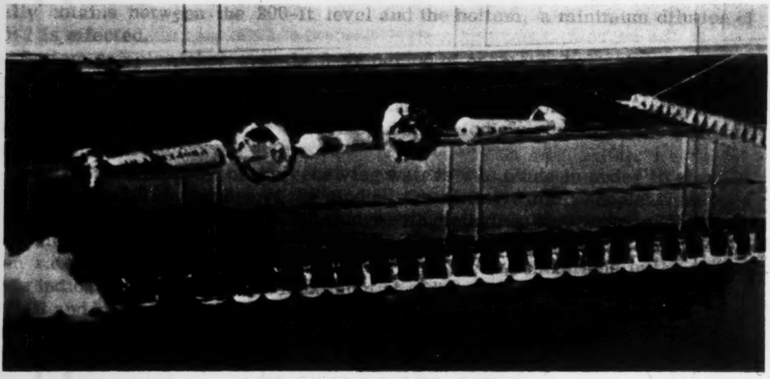

FIG. 10.—"PIG" USED FOR SLUDGE LINE CLEANING FLOATING IN FINAL TANK AT HYPERION

FIG. 11.—"PIG" ON DECK OF CITY OF LOS ANGELES OCEANOGRAPHIC SURVEY VESSEL, "PROWLER"

It is of interest to note that it was intended to recover the floating prover when it reached the ocean surface over the outfall terminus. Actually, currents moved the prover, which contained a sound signal that operated at 30-sec intervals, a sufficient distance during its ascent so that it was not seen. It subsequently washed up on the beach and was reported by bathers to be some sort of infernal machine. After appropriate examination by various ordinance experts, it was recovered by plant personnel.

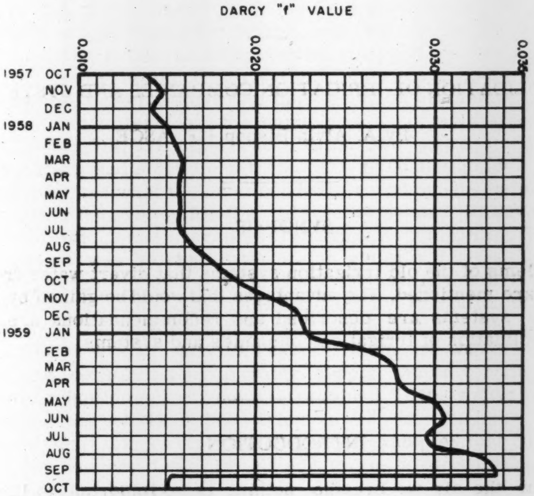

FIG. 12.—EFFECT OF SLUDGE LINE CLEANING UPON DARCY "f" VALUE

Fig. 12 shows the return of the Darcy "f" to about its original value. This cleaning of the sludge outfall will be continued on a routine basis at intervals to be established—perhaps one or two year intervals.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3177

CONSOLIDATION OF IRRIGATION COMPANIES AND SYSTEMS

By A. Alvin Bishop,[1] F. ASCE

SYNOPSIS

The problems of the old irrigation systems that divert water from a common source are mentioned. The advantages that would be gained by consolidation of these systems are described and recommendations are made concerning consolidation of irrigation companies and systems.

INTRODUCTION

Except for the air we breathe, nothing is so important to life as water, water for drinking, water for industry, water for all of the needs of man. Lack of water in the western United States is now restricting many activities. Agriculture founded upon irrigation is limited to the supply available. Industry must be located where water can be obtained. Cities are transporting water hundreds of miles to meet their domestic and municipal requirements. Competition for the water between agriculture, industry, and other users is becoming keen and reuse of unconsumed water is increasing. Agriculture, with its widespread irrigation, now consumes more water than all other uses combined and it will probably continue to be the largest single consumptive user of water. Along with the extensive use of water, irrigation probably is a major source of waste of the valuable water resource. This is due, in large measure, to the inefficiency of existing canals and distribution systems with

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Note.—Published essentially as printed here, in September, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2157. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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their duplication and obsolescence. The problems of these old canal systems must receive more public attention and emphasis.

For many years engineers have been concerned with the problems of small irrigation systems diverting water from a common source, serving the same general area and having common problems of operation and maintenance. In the early 1930's, O. W. Israelsen first proposed consolidation to four irrigation companies and four drainage districts in the West Millard county area of Utah, a consolidation that has not yet been achieved although the institutions involved have all the elements that would appear to dictate consolidation (common water supply, adjacent lands, common stockholders, duplicated canals, and common problems of operation, maintenance, and management).

In the early 1940's, two small irrigation companies in southern Utah were approached by the writer concerning their common problems. Both of these companies diverted water from the same small mountain stream. One had the direct flow rights, the other had a small storage reservoir. Over 60% of the stockholders had stock in both companies. To an outsider, it appeared that each company had what the other needed to round out its water supply and to make the irrigation system for the valley more secure. However, for many years these institutions had operated as two separate companies. The company with the reservoir was storing water in the winter and releasing it in early spring for the irrigation of the lands it serviced while the direct-flow company was diverting all of the high spring flows for the irrigation of their lands. By July, after the flood flows had passed, the direct-flow company was reduced to its low flow stage and the reservoir company was out of water. By combination and consolidation, a water supply, ample for the entire area throughout the irrigation season, could be provided. To the writer's knowledge, consolidation has not yet (1959) been achieved.

These examples, along with countless other examples that could be mentioned, make one ask why these obvious advantages of consolidation have been bypassed for so many years?

In order to answer this question, it is necessary to study the development pattern of our western water supplies. This development has to do not only with the physical development of canals, reservoirs, and other structures, but the legal, human, and economic developments as well.

PHYSICAL DEVELOPMENT

Beginning about 1850, the pioneer settlers located their diversions and canals low in the valleys adjacent to the lands easily irrigated and built their works accordingly. Later settlers were forced to make their diversions higher on the stream where construction of diversion works and main canals was more difficult. In addition, they were forced to select lands more distant from the stream requiring larger and longer canal systems. These systems were largely cooperative enterprises planned to solve the problems of the day for the individual or group. The ultimate development of the water supply of the area was not considered.

The result was inevitable; large areas of land were brought under irrigation without the benefit of our present day knowledge, and in many cases without the benefit of the engineering services that were available at the time. Later developments were seldom integrated into the existing systems, and although they were planned and constructed many years ago, they have maintained their identity down to the present time. Thus, many systems became established on a common water source with diversion dams for each system, parallel canals and duplicating works.

Many examples of parallel canals and duplication of structures can be pointed out. The system shown in Fig. 1 was originally constructed as a single canal, but within a few years it was reconstructed to form a "mounment to idiocy" that has endured for almost 100 yr. These canals have a common bank for almost 5 miles. The heavy growth of phreatophytes on the center bank is almost impossible to control. Enough muskrat holes probably exist between the canals to eliminate the initial purpose of dividing the water between the two groups of water users.

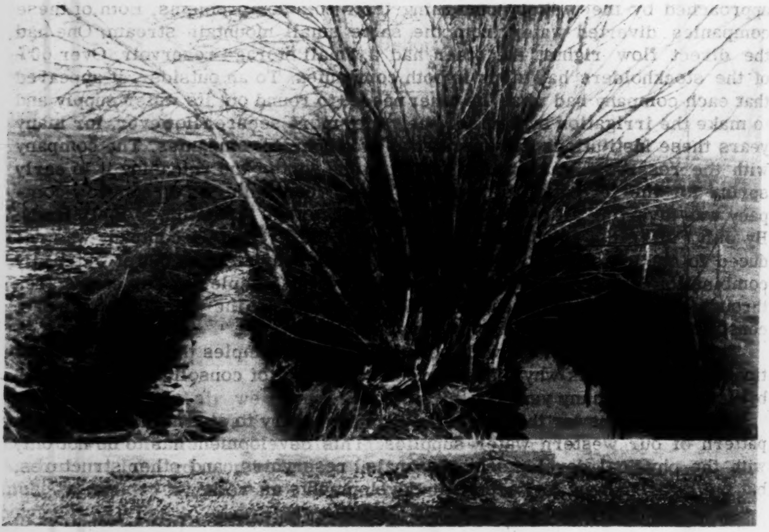

FIG. 1.—PARALLEL CANALS

This may be an extreme example of the evils of parallel systems, but it shows plainly the increased costs and inefficiencies that go along with duplicate works. The following problems will be found in areas where parallel and duplicate systems exist whether the canals are side by side or separated.

1. The costs of bridges and culverts is nearly doubled. Each road and highway crossing requires two bridges instead of one.
2. The amount of right of way required is increased and otherwise productive land must be dedicated to canal use.

3. Costly and inconvenient water diversions cannot be avoided. All of the deliveries are to lands below the canals and water diverted from the upper canal must bypass the adjoining system requiring costly and complicated structures.

4. Seepage losses are increased over the losses of a single canal.

5. Operation costs are greatly increased because of inconvenience and duplication.

6. Maintenance costs are also increased because of inconveniences and the impossibility of utilizing to full advantage the modern mechanical equipment now available.

LEGAL DEVELOPMENTS

Coinciding with the physical development of the water resources, was the legal development of the right to use water. Western states recognize the doctrine of appropriation, which simply stated is "first in time is first in right." The early pioneers who first developed the water obtained the first right to use of the stream, while later settlers acquired junior rights. Many of the original water rights are for direct flow only, while some of the later rights may combine storage rights with flood flow diversions.

The important point in the water right picture is that the mere possession of a water right may not guarantee any water to the owner of the right. When the water supply of a stream fails to satisfy the diversion requirements of existing water rights, the stream, of course, is over-appropriated and junior rights must give way to prior rights. This condition may happen only in dry years on some streams, whereas it may happen every year on others.

A good example of this situation can be obtained by superimposing the water right demand on the hydrograph of flow for almost any of the western streams. Charts similar to the ones for the Uintah River in eastern Utah will probably be the result. Fig. 2(a) is a daily flow hydrograph of the Uintah River for the year 1935 upon which has been superimposed the water rights of the stream. Although 1935 is a near normal water year, it will be noted that the water rights exceed the supply except for a short period during June when the flood flow caused by melting snow is at its peak. Prior rights during this year suffered only minor shortages before the snow melt season but junior rights were able to divert water only during the flood-flow period. Fig. 2(b) is a similar chart for the same stream in a low water year when the supply was not sufficient to satisfy even the prior rights. A court decree in this area, which limited the annual diversions to 3 acre-ft per acre, produced the variation in the seasonal water right picture. Ordinarily the water rights may be represented by straight horizontal line on the chart for the period of use.

Consolidations of irrigation companies will certainly bring about a meeting of water rights of different priorities. The evaluation of one right in terms of another will be most difficult, for in a good water year a junior right may obtain as much water as a prior right, but in drought years, the junior right may get no water at all. Any water right evaluation must have as its foundation a complete analysis of the water supply. After consolidation, the water rights must maintain their identity to satisfy legal requirements. Many companies,

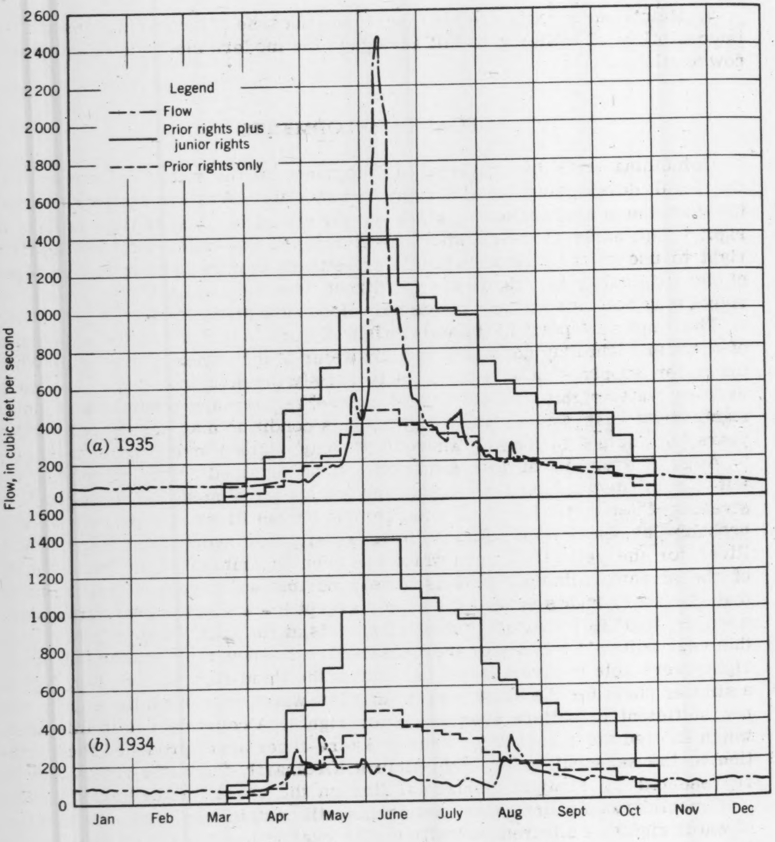

FIG. 2.—UINTAH RIVER HYDROGRAPHS

however, obtain their water supply under several separate water rights and this feature of maintaining the identity of the right creates no problem.

DEVELOPMENT OF HUMAN PROBLEMS

A complicating factor in the combining of irrigation systems is the attitude of the present owner. He has a special relationship to water rights that he developed. He has had to guard it jealously for fear of losing it. He has adapted his farming to the water supply represented by it. He will probably resist any combinations because of the uncertainity of the result. He knows what he can expect from what he has. This is true whether the owner is an irrigation company or an individual.

Changes and combinations may require changes in farming practices and it is only natural for owners of water rights to resist change. The development of the water supply and water rights has sometimes developed jealousies and hard feelings against adjacent water users. Although the original settlers may in many cases be dead, the antagonisms, fears, and jealousies of the original pioneers have been passed on to their heirs and successors, and the problem has remained alive. Thus, a deep seated attitude towards the existing system has developed that will increase the problems of consolidation.

ECONOMIC DEVELOPMENT

A fourth development is the economic condition of the existing systems. In considering the economics of the old irrigation system, one must recognize the fact that most of the development work was done by the owners of the lands benefited. The early settlers diverted water directly from the streams by means of individually constructed dams and ditches that were planned and built for the purpose of solving their individual irrigation problem. The irrigation works constructed by individuals or small groups were considered private property, subsequent developments were seldom combined with existing systems. The resulting developments, in many cases, are debt free. Original construction charges have been repaid and the present cost of water to the users is for operation and maintenance of the system. In a few instances, a group may have an active betterment program to improve their system.

Many of these old systems still maintain their initial usefulness and efficiency, but even so, they may have grown old and outdated. No matter how wise their initial conception, nor how sound their design and construction, irrigation systems deteriorate and modern technology and economic conditions overtake them. The standards of usefulness have changed greatly since the turn of the century.

A good example of this is the horse-drawn scraper. In its day this was a very efficient piece of equipment for building canals and grading land for irrigation. In 1920 it was in common use by all farmers. The same scraper today, even in perfect condition, would be very inefficient. The scraper has not changed, it will do the same work at the same rate as it would 40 yr ago, but by comparison with modern equipment, it would be considered inefficient and obsolete. In a similar manner, many of the canals are inefficient and obsolete, resulting in inefficient use of water, poor distribution and delivery practices, duplicating and parallel canal systems and other inadequate features.

IRRIGATION SYSTEMS IN THE WESTERN UNITED STATES

Some idea of the extent of the problem can be obtained by comparing the development of communities with the development of irrigation systems. During the pioneer development period, settlements were formed on the streams where water supplies were available. Even the smallest creeks have a small community at their mouths and much of the water for irrigation in the west comes from small mountain streams. Utah now has nearly 400 cities, towns, and villages as the result of this development. By contrast, however, the 1950 census lists 3,165 irrigation enterprises in Utah, 1,058 of which are group enterprises. Of these group enterprises in Utah, 406 are unincorporated mutual companies, 634 are incorporated mutual companies, 5 are irrigation districts, 2 are commercial companies, 7 are cities, 3 are irrigation developments of the United States Bureau of Indian Affairs and one is a Bureau of Reclamation Dept. of Interior (USBR) operated project (Other USBR projects in Utah are operated by Water User's Associations). The area irrigated by the 1,058 group enterprises in Utah is 1,042,497 acres making the average 985 acres per enterprise. The 2,107 remaining projects are listed as single farm enterprises consisting largely of wells and pumping plants developed for individual farms.

For the 17 western states the 1950 census lists 111,940 separate irrigation enterprises, 101,770 of which are single farm enterprises and 10,170 group enterprises. Of these group enterprises 6,417 are unincorporated mutual companies, 2,880 are incorporated mutual companies, 131 are commercial companies, 483 are irrigation districts, 37 are USBR projects, 141 are Bureau of Indian Affairs enterprises, 31 are state enterprises and 50 are city operated. The 10,170 group enterprises in the 17 western states irrigate a total area of 14,713,888 acres or an average area of 1,447 acres per enterprise.

Not all of these enterprises, of course, could be considered to be badly in need of improvement and only a small percentage might be classed as duplications where consolidations would be recommended. It should be pointed out, however, that if as many as 10% of the group enterprises fell into this class it would mean that more than 1,000 systems should be consolidated and modernized.

In 1946 Israelsen and others published[2] the results of a survey of irrigation companies in Utah. Data were obtained from 688 separate companies. There is no legal limit to the minimum size of an irrigation company as seen by the fact that 179 of the 688 companies serve areas of less than 300 acres, some even less than 100 acres. The other 509 serve areas larger than 300 acres, the largest one of which serves approximately 50,000 acres.

To staff the 688 separate companies in Utah requires the services of 2,606 officials. Although water delivery and distribution is considered to be largely an engineering problem only 69 of the 688 companies regularly employed an engineer. The importance of the legal aspect of water rights is evidenced by the fact that 167 of the 688 companies regularly employed attorneys.

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[2] "Irrigation Companies in Utah, their Activities and Needs," by O. W. Israelsen, J. Howard Maughan, and George P. South, Utah Agricultural Experiment Station Bulletin 322.

The large number of irrigation enterprises and the small area served by group enterprises indicates that there is a problem of considerable magnitude concerned with consolidation of irrigation companies and systems.

CONSOLIDATION PRESENTS CHALLENGE

The physical development of the canal systems and appurtenent works, the legal development of the right to use water, combined with the human and economic problems have created the present predicament with regard to the older irrigation systems. The dilemma is very real. In the interest of obtaining maximum benefits from the valuable water resource, something must be done to consolidate, revamp, and modernize these old systems. However, the reorganization and betterments will certainly call for an initial outlay of capital. To solve these problems, a major challenge is presented to the engineers, lawyers, economists, and water users interested in water resources. How can this challenge be met and what is necessary to get the job done? The writer recommends four basic steps or activities as follows:

First Step.—There must be a recognition of the problems of the old irrigation systems by public and private groups dealing with the development, use and administration of the water resource. This includes the irrigation company officials and irrigators, Federal and State Agencies, consulting engineers, and lawyers.

Second Step.—There must be an education process to create a desire to consolidate and improve the existing irrigation companies and systems.

Third Step.—There must be a sound consolidation plan for systems to be integrated. This plan must be considered from an engineering standpoint, legal standpoint, and economic standpoint.

Fourth Step.—There must be provided a method by which necessary changes can be financed.

In order to adequately present the facts to the water users involved, a careful appraisal must be made of each individual system. The existing physical facilities, water rights, water supply, legal problems, and special problems must be inventoried. Because the water supply is the foundation of the enterprise, an evaluation of the water supply with respect to its occurrence in time and amount is necessary in order to evaluate the comparative desirabilities of the systems to be integrated. Hydrologic studies must be made where data are not available and where water supply information is available, it must be carefully studied and evaluated.

Potential water supplies must be investigated, such as storage, groundwater development, and transdiversions. Methods must be found for improving the efficiency of existing water conveyance and use through canal lining or changes in administrative procedures. Methods of delivery in use by the companies must be studied in relation to the water supply in order to determine if changes might be made that would be advantageous. In short, all of the facts connected with the water resource must be obtained.

The physical facilities of the irrigation companies must be appraised to determine their adequacy and value. Where duplications and overlapping systems are evident, designs, and cost estimates for a comprehensive system must be made. Existing structures should be studied to determine their adequacy based on present day standards. Additional devices needed to control and measure the water efficiently should be determined. Surveys to improve canal alinements must be made. The operation costs of the existing facilities along with the maintenance problems must be evaluated. Costly operation or maintenance practices that will be eliminated by consolidations might be discovered. This is an important feature and may be a means whereby the engineers and others can obtain cooperation towards consolidation from the water users involved.

The legal problems will be many. The relation of the water supply to the water right and need and the method of combining the water rights for the integrated companies must be established. The identity of the water right must be maintained, but the water represented by the water right must be combined and redistributed according to requirements of the water users under the combined system. The pooling of stock having different priorities or different basic values must be worked out and stock re-issued having an equal par value and representing the same quantity of water per share. The rights of way problems involved in new construction will be complicated.

Finally, the economics of consolidation must be clearly outlined. The cost of construction and the benefits must be determined. The savings of water affected by eliminating overlapping systems must be evaluated in terms of the savings anticipated and the cost of the construction required to bring about the savings. Savings in water, savings in operation, and savings in maintenance must all be evaluated in terms of dollars. Costs of new construction, costs of new programs, and changes necessary to modernize the system must be carefully estimated. The financial conditions of each company must be determined and debt obligations liquidated or adjusted within the framework of the consolidation proposed. The economics of the area and the economics of the country must also be given consideration. To prove the value of consolidation, a consideration of all economic features must leave the benefits exceeding the costs.

These particular studies become more involved when it is realized that each combination or consolidation presents unique problems. The situation cannot be studied in one area and solutions applied to other areas as a blanket rule. In every consolidation scheme, the merits of the consolidation must be considered individually. Thus, every area where duplicating systems exist, must be given the same detailed consideration with regards to engineering facilities, economic factors, legal implications of water rights and rights of way in the benefits to be derived from the consolidation.

After all of the field surveys have been made, the water supplies determined, the water rights evaluated, the economic analysis carefully prepared, the greatest problem still remains. That of harmonizing the conflicting interests of the groups involved into a unified purpose. This involves the delicate handling of the human problems that have developed and confounded most of the consolidation attempts in the past. One would think that a preponderance of economic evidence and education of the groups concerned would simplify this problem. However, many of the irrigators will maintain a skepticism in the face of overwhelming economic evidence. Their stubborn portion has often led to the dire prediction, "Before consolidation of these irrigation companies can be achieved, there must be several funerals." In some cases, the funerals have long since occurred, yet the consolidation has not yet been completed. The prejudices and jealousies of the leaders of the companies were passed on to their successors and are still evident, remaining as a major obstacle in the path of any consolidation attempt. In such cases, consolidation must wait until economic pressure or public sentiment forces the irrigation companies to utilize their water supplies more wisely and efficiently.

NEW PROJECTS OFFER PROMISE FOR CONSOLIDATION

The greatest possibility for consolidation will probably be realized in the construction of new facilities of major replacements of existing structures. Where some major project involving all concerned can be designed, a common ground is established to bring conflicting interests together. This offers an excellent opportunity to unite all groups involved under the new project. Where such projects are now under construction by public agencies or private groups, the possibility of combining small irrigation companies into larger more efficient organizations should not be bypassed even though consolidation would require considerable time in negotiations, education, and legal work. Storage reservoirs, regulation reservoirs, diversion dams, transdiversions to develop new water, and canal lining projects all offer possibilities in this regard.

In two instances in Utah, small companies diverting water from the same stream have been consolidated by such a means. In one case, a small storage reservoir was constructed by the irrigation companies and after the construction of the reservoir one enterprise emerged to combine the new water made available through the project with existing supplies and to distribute it to the water users. On many of the USBR projects the same prospect is in evidence, if properly handled. The large multiple purpose projects and the large area served by the reclamation works offers an excellent opportunity to unite many small irrigation enterprises together in a district for the distribution of the water under the new project. At the same time, the consolidation of the rights, water supplies, canal systems, and so on, of the duplicating enterprises should be developed. The prospects would be better at this time than at any other opportunity afforded. To attempt to consolidate merely by compiling overwhelming evidence concerning the advantages, will not completely do the job. More success will be obtained by combining the idea of consolidation with some project that will unify the interests of the water users toward a common goal.

INTEREST GROWING IN CONSOLIDATION PROBLEMS

In 1950 the Irrigation Division of the American Society of Civil Engineers established a Committee on Consolidation and Progressive Betterment of Old Irrigation Systems. This was perhaps the first organized approach to the problems of consolidation or irrigation companies and systems. The objectives of the committee were defined as follows:

1. To develop basic principles upon which the consolidation of irrigation enterprises using water from a common source and having common problems can be proposed.
2. To develop methods and procedures for carrying out progressive betterment or major improvement programs on old irrigation systems within the financial limitations of the enterprise over a period of years.

3. To consider the problem of rehabilitation of irrigation systems that were built many years ago and without the benefit of engineering services.

The committee had done considerable work but because of the complexity of the problem, progress is slow. Other organizations are now becoming interested in this problem. To add momentum to the consolidation crusade, it may be desirable to recommend that federal and state agencies develop some inducements to encourage consolidation and betterment of irrigation systems. These inducements could take the form of (1) financial subsidy by making non-interest bearing loans, (2) providing engineering and legal assistance, and (3) cost sharing as in the case of the Agricultural Stabilization program.

Public recognition and public support of the problems of consolidation of irrigation systems are necessary in addition to the services of engineers and attorneys dealing directly with the problem. Until this can be achieved, progress will continue to be slow.

ADVANTAGES OF CONSOLIDATION

Where consolidation can be achieved, the writer believes the following advantages will result:

1. Increased efficiency in the use of the existing water supply by eliminating duplicate systems and making other improvements;
2. Increased efficiency in the management of irrigation enterprises by reducing the total number of water masters, board members and other officials required;
3. Increased efficiency in the operation of irrigation systems by having projects of sufficient size to justify the employment of technically trained engineers and others;
4. Increased safety in operation by modernizing and improving the physical facilities;
5. Increased flexibility of the water supply to meet the agricultural requirements; and
6. Increased strength and effectiveness of the irrigation institution.

Consolidation of irrigation systems offers a fertile field for the more efficient utilization of water resources. In relating this problem (meeting of Irrigation Division of the American Society of Civil Engineers in Salt Lake City in 1954) George D. Clyde, F. ASCE had this to say,

> "Consolidation of small irrigation companies offers great opportunities to increase the net usable water supply available for irrigation and presents a challenge to the ingenuity and ability of the engineers, the progressiveness of the water users and the ability of the legal profession."

CONCLUSIONS

Small irrigation systems diverting water from a common source and built over a period of many years have developed a number of problems that might be solved by consolidation of these systems. The problems may be divided into four types as follows:

(1) Problems concerned with the physical situation involving parallel canals, duplicate structures, multiple diversions and costly operation and management;
(2) Legal problems concerning the right to the use of water involving complex combinations of priority, period of use and water supply;
(3) Human problems involving the attitude of water users toward the development and use of water; and
(4) Economic problems connected with the physical development and value of the water supply.

Recognition and solution of the problems that can be solved by consolidation presents a challenge to engineers, lawyers, economists and to the irrigation enterprises involved but offer a fertile field for increasing the efficiency of water use and a means to obtain maximum benefits from the limited water resource. There are many advantages that may be derived from consolidation and the writer believes that the new project offers the best opportunity to establish a basis to bring conflicting interests together.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3179

DIFFUSERS FOR DISPOSAL OF SEWAGE IN SEA WATER

By A M Rawn,[1] Hon. M. ASCE, F. R. Bowerman,[2] M. ASCE, and
Norman H. Brooks,[3] M. ASCE

With Discussion by Messrs. J. M. Jordaan, Jr; C. H. Lawrance; and A M Rawn,
F. R. Bowerman, and Norman H. Brooks

SYNOPSIS

For large sewerage systems the effectiveness of sewage disposal by dilution in the ocean depends on efficient dispersal of sewage into the receiving waters. By the use of large multiport diffusers on the ends of their outfalls, the Los Angeles County Sanitation Districts have, in recent years, greatly improved the characteristics of the ocean water near the disposal site despite steadily increasing discharges of primary effluent.

This paper includes a history of the ocean outfall system operated at Whites Point, Calif., by the Los Angeles County Sanitation Districts; an outline of certain techniques developed to predict dilutions of sewage effleunt discharged in sea water; a description of the use of multiport diffusers for improving initial dilution; and a procedure for successful hydraulic design of large diffusers.

INTRODUCTION

The disposal of sewage effluent to bodies of water for dilution and final disposal requires an effective mingling of the effluent with the water body to

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Note.—Published essentially as printed here, in March, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2424. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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achieve the following purposes: (1) the effective oxidation of the fine suspensoids and dissolved organic compounds; (2) the reduction of bacterial population; (3) the prevention of odor nuisance; and (4) the removal of particles by sedimentation with natural detrital bottom materials.

The disposal of sewage effluents into water bodies involves hydrodynamic relationships relating to the dispersal of one fluid into another; increased complexity results in the ocean from the substantial differences in fluid densities which cause the effluent to rise toward the surface of the salt-water body. For reasons to be cited subsequently, there are significant advantage in dispersing sewage effluent near the floor of the ocean as a means of increasing initial dilution. Some of the concepts embodied herein are applicable to the disposal of sewage effluent to freshwater bodies; however, disposal to the ocean is the primary subject of this presentation.

Where the favorable exchange of water through tidal or current movements exists, primary sewage treatment will usually afford adequate protection to the receiving waters, often without the necessity to chlorinate. Since primary treatment involves sedimentation processes which remove floatable constituents of sewage as scum from the surface of the sedimentation tanks while at the same time removing readily settleable solids from the bottom of the tanks, it can be seen that the primary treatment process eliminates the common complainst voiced against sewage disposal by dilution, to wit, the building of bottom sludge beds or the fouling of surface waters with floatable debris. No such similar protection is afforded by fine screening plants since these do not separate material according to their specific gravity, but only as to size. Floating rafts of fine sewage debris will be the almost inevitable result of the disposal of finely screened effluent into a body of ocean water.

The security of recreational ocean waters with respect to public health may usually be guaranteed through disinfection by artificial means such as chlorination, or by designing the outfall so as to prevent sewage effluents from reaching bathing areas in dangerous concentration. Where sufficient oxygenated diluting water is available to achieve initial dilutions of, say, 50 or more, there appears to be no need for more reduction of organic content of the sewage effluent than that provided by primary treatment, in order to prevent odors or other septic nuisances. Such dilutions are often found available where primary sewage effluent is proposed for discharge to ocean bodies.

OUTFALL SYSTEM OF LOS ANGELES COUNTY SANITATION DISTRICTS

The performance of a particular outfall or system of outfalls must be evaluated in terms of the physical circumstances extant at the site of the installation. No direct comparisons can, or should, be drawn between ocean outfalls unless consideration is given to the similarities and differences in the systems so compared.

Site Characteristics.—The Los Angeles County Sanitation Districts' outfalls extend off Whites Point, a small promontory on a peninsula of land extending into the Pacific Ocean southwestward from Los Angeles. The outfall site was chosen subsequent to work done by District engineers in the early 1920's; however, 6 miles of tunnelling under a range of low mountains was required to reach the shoreline, so that the first ocean outfall was not commenced until 1935 and not completed until 1937. In 1958 the outfall system was disposing of the sewage from an area of almost 600 sq miles with a population of more than 2,500,000. Characteristics of the outfall site are as follows:

1. The rocky Palos Verdes Peninsula, comprised largely of shales and schists, juts about 5 miles into the Catalina Channel from the Southern California coastline, terminating at its most offshore face in a cliff which rises a distance of 80 ft or more abruptly from the ocean's edge.
2. The bottom, offshore from the peninsula, extends seaward at a slope of about 1 in 50 as a wave-washed ledge of rock, becoming sandy at a water depth of about 30 ft or 40 ft, with detrital muds commencing at a water depth of about 80 ft to 90 ft, remaining as mud bottom further offshore into the Catalina Channel.
3. Strong littoral currents sweep the promontory, running predominantly parallel to the shoreline, with occasional weak currents of short duration moving toward shore.
4. Weak winds, usually from the west, cause movement of surface water toward shore, but have as their principal effect the establishment of a mass transport of water such as to induce upwelling of cold bottom water near the shore. The favorable effect of this latter phenomenon will be investigated subsequently.

The Outfall System.—At Whites Point, the Los Angeles County Sanitation Districts have constructed three ocean outfalls; one built in 1937, 60 in. in diameter and extending 5,000 ft from shore to a depth of 112 ft; a second, built parallel to the first, in 1947, having an internal diameter of 72 in., originally terminating the same distance from shore, but later lengthened to 6,800 ft from shore in 1953, at which time a diffusion structure was also constructed on the line; and a third, 90 in. in diameter, completed in 1956, extending 8,000 ft from shore to a depth of 212 ft, terminating in an even more efficient diffusion structure than that placed on the 72-in. outfall. The hydraulic capacity of the three pipes is about 500 mgd. The outfall system is designed for disposal of primary effluent, depending on natural dispersion and oxidation in the receiving waters. The outfalls are shown in Fig. 1 and 2.

The three ocean outfalls have been built as trenched conduits through the rock section of the ocean's floor, becoming semiflexible, articulated pipelines laid on the surface of the sand and mud at greater ocean depths.

All three lines have been constructed from cast reinforced concrete pipe with meehanite cast iron tongue-and-groove or ball-and-socket joints. Examination of the 60-in. pipe in 1957 showed that the cast iron had corroded at the rate of about 10 mils annually, so that penetration of corrosion (that is, graphitization) of about 0.2 in. has occurred during the approximately 20 yr of submersion. No significant loss in strength has yet been occasioned, although continued graphitization at the same rate may cause joint failure in another 40 yr to 50 yr. The most recently placed outfall has sacrificial magnesium anodes attached to the cast iron end rings. Concrete cores bored from the 60-in. diameter outfall show that the crushing strength of the concrete exceeds that measured more than 20 yr ago.

Effluent Disposal Without Diffusers.—In 1933, the Districts commenced a routine observation of the ocean environment and measurement of bacterial populations. An intensified survey program was commenced in 1949, where, in addition to visual observations and bacterial analyses, the following factors were measured:

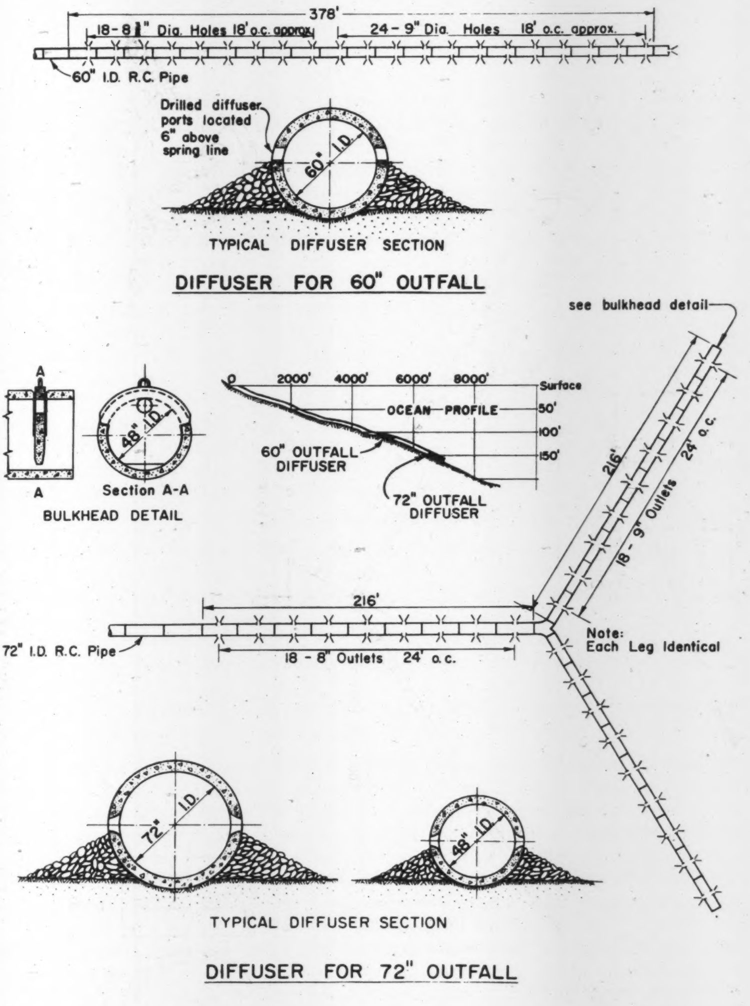
378'
18 - 8[illegible]" Dia. Holes 18' o.c. approx.
24 - 9" Dia. Holes 18' o.c. approx.
60" I.D. R.C. Pipe
Drilled diffuser ports located 6" above spring line
60" I.D.
TYPICAL DIFFUSER SECTION
DIFFUSER FOR 60" OUTFALL
see bulkhead detail
A
A
48" I.D.
Section A-A
BULKHEAD DETAIL
0
2000'
4000'
6000'
8000'
Surface
OCEAN PROFILE
50'
100'
150'
60" OUTFALL DIFFUSER
72" OUTFALL DIFFUSER
216'
24' o.c.
18 - 9" Outlets
216'
72" I.D. R.C. Pipe
18 - 8" Outlets 24' o.c.
Note:
Each Leg Identical
72" I.D.
48" I.D.
TYPICAL DIFFUSER SECTION
DIFFUSER FOR 72" OUTFALL

FIG. 1.—OCEAN OUTFALL DIFFUSERS OF THE LOS ANGELES COUNTY SANITATION DISTRICTS

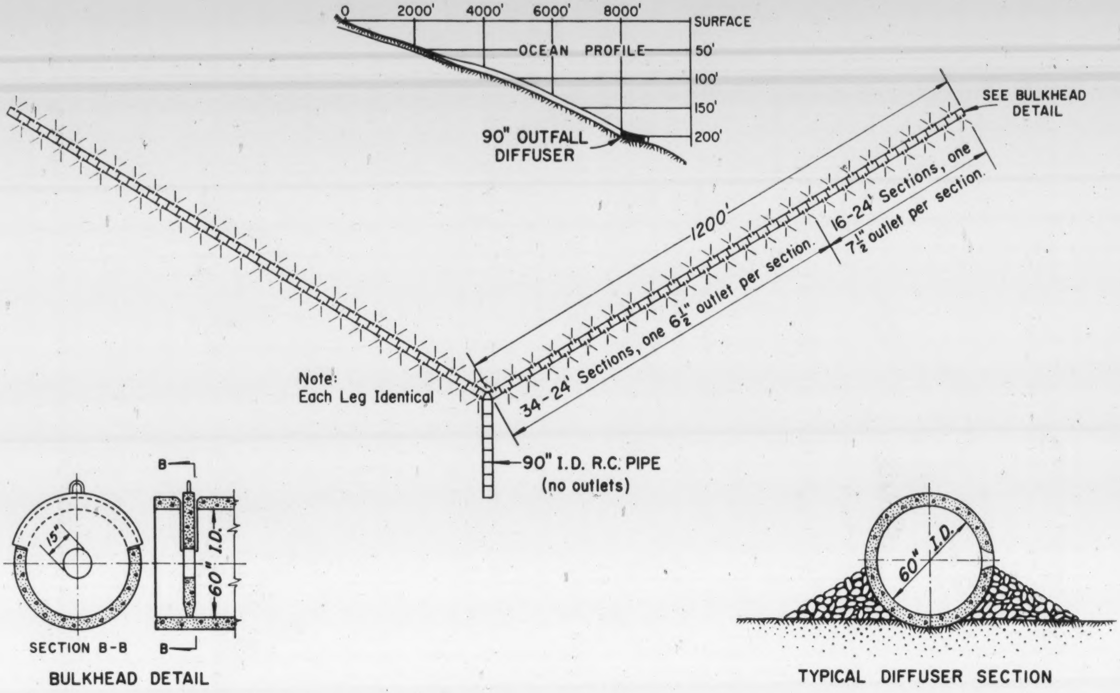
0
2000'
4000'
6000'
8000'
SURFACE
OCEAN PROFILE
50'
100'
150'
200'
90" OUTFALL DIFFUSER
SEE BULKHEAD DETAIL
1200'
16-24' Sections, one 7½" outlet per section.
34-24' Sections, one 6½" outlet per section.
Note: Each Leg Identical
90" I.D. R.C. PIPE (no outlets)
15"
B
B
60" I.D.
SECTION B-B
BULKHEAD DETAIL
60" I.D.
TYPICAL DIFFUSER SECTION

FIG. 2.—DIFFUSER FOR 90-IN. OUTFALL OF THE LOS ANGELES COUNTY SANITATION DISTRICTS

1. Ocean currents in the vicinity of the point of discharge at the surface and at a depth of 10 ft;
2. Wind velocity; and
3. Water temperature at the surface and at various depths at two points, one 5,000 ft and the other 7,500 ft offshore along the line of extension of the outfalls.

The level of bacterial activity is seen in Fig. 3 to be directly related to the amount of sewage effluent; in Fig. 3 the bacterial concentrations are based on the percentage of bacterial counts (coliform M.P.N.—as defined in "Standard Methods for the Examination of Water and Waste Water," American Public Health Association, 11th Edition) exceeding 10 per ml. California regulations establish for bathing waters a limit of 10 coliform organisms per ml as the level beyond which public health and safety may be impaired, qualified by the stipulation that the frequency of counts greater than 10 coliform per ml may reach, but not exceed, 20% of the number of samples taken, based upon twenty consecutive samplings. It is presumed that the occurrence of coliform counts exceeding 10 per ml at a greater frequency than 20% of the number of readings is sufficient to establish some credence in the data. By 1952, bacterial levels were approaching the point where improvements were indicated in the means of disposal at Whites Point (Fig. 3) and it was indicated that further thought should be given to improving the mixing of sewage with sea water at the point of discharge and to exploring design modifications which might improve conditions at the outfall site. The sewage field resulting from the discharge from the then open-ended pipes was objectionable for the following reasons:

a. Surface layers of poorly diffused sewage sometimes traveled with shallow, wind-driven ocean currents, which move predominantly shoreward;

b. Mixtures of sewage and sea water, often exceeding 2% sewage, were quite stable and maintained identity as "sewage fields" for extended periods of time and for several miles of travel;

c. Shoreline coliform counts at times exceeded allowable limits;

d. Visible sewage fields created areas of discolored water seen from the shore; and

e. Poorly diffused sewage effluent created odor problems, both in and near the floating field.

Effect of Ocean Temperature Gradients.—Experience at Whites Point showed coliform concentrations at the shore in the vicinity of the outfalls to be higher in winter than in summer. The difference was due, in part, to stronger shoreward winds occurring during winter months, but in the main, seemed to be influenced greatly by the stronger ocean-temperature gradients in the summer.

Sewage effluent discharged from a deep ocean outfall mixes first with cold bottom water. It was often observed that sewage-sea water mixtures reached the ocean surface some 10°F colder than nearby surface water. At times, the colder mixture of sewage and sea water was sufficiently more dense than surface ocean water that, on reaching the ocean surface, it would resubmerge and spread laterally below the surface, a phenomenon readily demonstrated by dumping fluorescein dye into the rising column and observing it spread laterally and disappear under nearby surface water. Since the phenomenon just described appeared at times coincident with lower bacterial counts at shore sampling stations, both effects being observed during the summer and fall, it

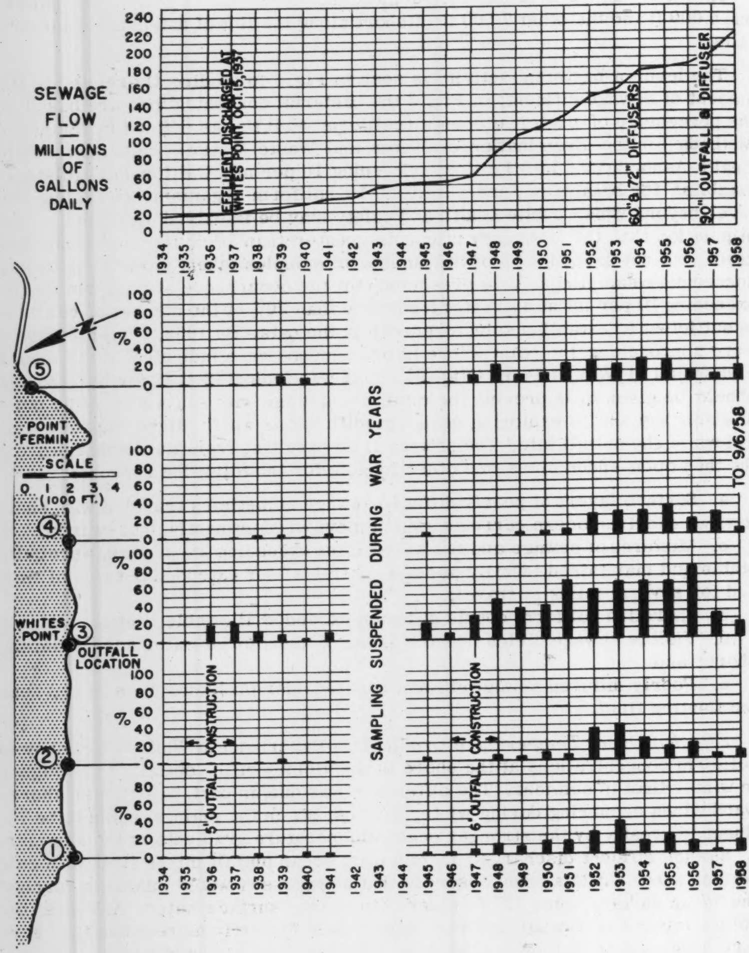

FIG. 3.—ANNUAL PERCENTAGE OF COLIFORM COUNTS EXCEEDING 10 PER ML AT SHORE STATIONS

was deemed advisable to measure more carefully some of the physical properties of the ocean environment in search of a solution.

To the weekly measurement of temperatures at two specific locations offshore was added the measurement of surface and subsurface temperatures at 1,000-ft intervals from shore along the line of extension of the outfalls to a distance of 10,000 ft seaward. Temperatures were measured with a highly sensitive thermistor mounted on a sounding cable and the data plotted on profile sheets (Fig. 4). The presence of cold bottom water at shallower depths during the summer and fall is accounted for by upwelling of bottom water along the Southern California coast, resulting from the prevailing westerly winds. Warm surface waters are quite naturally explained during the summer and fall as a result of the greater insulation. The winter-spring regime is characterized by very little change in water temperature from surface to bottom in nearshore water, in part because of the absence of upwelling, but also because the shallower waters are more completely mixed by the stronger wave action. The deep-water diffusers, built by the Sanitation Districts at the ends of their ocean outfalls, have been designed to invade the area of colder bottom water in order to create submerged fields of dilute sewage effluent.

Studies of Diffusers.—Small-scale model studies confirmed theoretical conclusions that a multiple-port diffuser system was superior to other systems tested. Preliminary calculations demonstrated that a multiple-port diffuser could lie on a seaward sloping ocean floor and flow full through all ports at all conditions of flow without costly penalty in hydraulic head loss.

In July 1953, with the consent and cooperation of the East Bay Municipal Utility District, the writers studied the operation of the Oakland Outfall Diffuser, located in the ship channel between Oakland and Yerba Buena Island in San Francisco Bay. The average depth to the top of the horizontal, single-barreled, multiport diffuser is about 40 ft. Digested sludge is discharged with the sewage effluent for dispersion in the bay. Because of high turbidity, the diver inspected by feel, with zero visability. Inshore sections of the diffuser evidenced about 1/4 in. of "greasy slime" around the inside of ports, with the deposit lessening toward the offshore end of the diffuser, indicating that most of the flow had been passing through the innermost ports. The buildup of sediment, grease, and slime was very limited, causing no concern over clogging of the pipe, even though the pipe had been in service for the discharge of primary effluent and digested sewage sludge for about 1 yr. Recent observations on the Sanitation Districts' outfall diffuser have verified these earlier findings.

Construction of Diffusers.—Based on theoretical analysis and the operating structure at Oakland, diffusion structures were designed and placed on the Districts' existing 60-in. outfall and 72-in. outfall. The 72-in. outfall was discharging a flow of about 100 mgd 5,000 ft seaward and it was deemed advisable to convey that quantity of sewage farther from shore and to greater depth. An additional 1,500 ft of 72-in. diameter pipe was added to the outfall, extending the structure to 6,500 ft seaward in 150 ft of depth. Fig. 1 shows the design of the 60-in. diffuser and the 72-in. diffuser.

A single-barreled diffuser was deemed adequate for the 60-in. outfall, discharging 50 mgd at 100-ft depth. The original three-outlet cast-iron diffuser placed at the seaward end of the 60-in. outfall was closed with steel plates, leaving only a small opening at the top and bottom of the central port to provide some self cleansing. Commencing with the first inshore pipe section from the

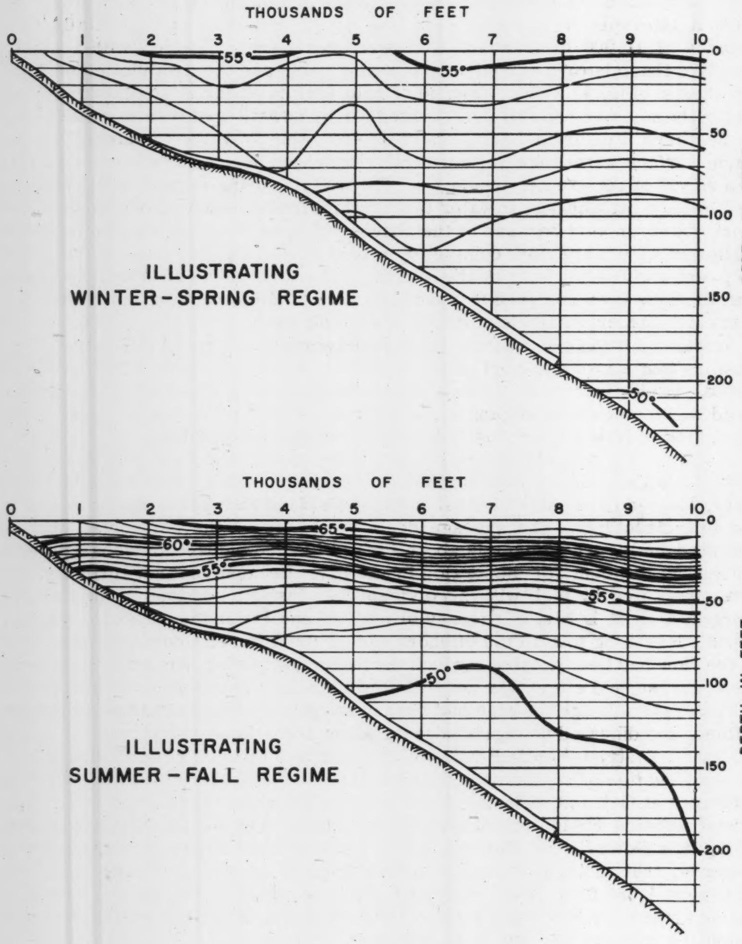

FIG. 4.—OCEAN-TEMPERATURE PROFILES AT WHITES POINT

cast iron diffuser, 9-in. diameter holes were bored slightly above the spring line on opposite sides of the pipe, two in each 18-ft pipe section (Fig. 1). The additional head loss for the diffusers did not exceed 2 ft at peak flow. The operational experience gained from the two diffusers during 1954 and 1955 provided confirming data for the multiport diffuser to be used on the next proposed outfall.

In November 1956, the Districts completed and placed in service the third of its ocean outfalls, 90-in. inner diameter, extending seaward 8,000 ft to a depth of 212 ft; Fig. 2 shows details for this diffuser. The total cost for construction of the 90-in. outfall and its diffusion structure was $1,686,000; it has a design capacity, based on dispersing efficiency, of about 150 mgd mean flow. A summary of the physical characteristics of the three outfalls and diffusers is shown in Table 1.

Inspection and Maintenance of Diffusers.—Following a year of discharge of the primary effluent thru the 60-in. diffuser and the 72-in. diffuser, a physical inspection was made of the condition of the ports to determine if sedimentation was occurring in the main barrel of the pipe or if the ports were clogging. Inspection showed all ports to be flowing; side ports in the terminal pipe section were approximately half filled with "soap-like" deposits which partially blocked the flow, but were readily dislodged. The bottom half of the terminal pipe section was filled with a deposit which tapered so rapidly that it was not discernible in the next pipe section. There was no flow from the bottom slot in the end gate; however, sewage effluent was found to be flowing from the hole in the upper end of the gate. The observed deposits were found only in the last pipe section and it was not deemed necessary to clean the diffuser at that time.

After about 4 yr of operation, a careful inspection (by free-swimming divers using self-contained underwater breathing apparatus) showed that the deposition of settleable material had reached the third port from the terminal of the legs of the 72-in. line and the diffuser was cleaned. Using free-swimming divers, the bulkheads which closed the ends of the legs were jacked partially open and sufficient sewage effluent passed thru the diffuser to eject the settled solids out onto the ocean floor. The floatable material blown from the pipe was collected at the ocean surface to prevent its drifting onshore. No further action was necessary to satisfactorily scour the debris out of the diffuser. The diffuser on the 90-in. outfall was similarly cleaned without incident; a routine program of flushing every 2 or 3 yr appears to be a simple, relatively inexpensive means of cleaning the diffusers.

Evaluation.—The experience gained from almost 5 yr of operation of manifold diffusers located at considerable depth in an ocean environment having regular and rapid change of water has clearly demonstrated certain ameliorating effects resulting from such diffusers when compared with previous poorly diffused discharge. One very desirable effect has been a reduction in the bacterial population in near-shore waters. The most dramatic changes occurred in the area immediately above the point of discharge. Whereas, well-defined "boils" were readily distinguishable before adequate diffusion was initiated, it has become difficult to detect the location of the sewage discharge by other than chemical or bacteriological means, except on infrequent occasions during the winter when discolored sea water appears over the diffusers during periods of high flow. Oceanographic studies made prior to the use of diffusers could readily depend on the sewage bubbles as points of reference; with the diffusers installed, it has become necessary to establish off-shore locations by use of sextant, as only in this manner can one be assured of sampling from an area directly over the point of discharge, or at some predetermined distance from the outlet.

TABLE 1.—SUMMARY OF DIFFUSER CHARACTERISTICS

	Outfall		
	1	2	3
Outfall inside diameter, in inches	60	72	90
Year outfall placed in service	1937	1947	1956
Year diffuser placed in service	1954[a]	1953	1956
Length of ocean outfall excluding diffuser, in feet	4,500	6,400	7,900
Total length of diffuser, in feet	384	648	2,400
Average depth of discharge (below mean sea level), in feet	108	155	203
Number and diameter of ports	26 at 9 in.[b] 18 at 8.4 in.	40 at 9 in.[b] 18 at 8 in.	2 at 15 in. 32 at 7.5 in. 68 at 6.5 in.
Shape of ports	circular (sharp-edged)	circular (bell-mouthed)	circular (bell-mouthed)
Ratio: $\frac{\text{Total port area}}{\text{Outfall cross section}}$	0.94	0.85	0.63
Spacing of ports, in feet	2 every 18 ft	2 every 24 ft	24
Design peak flow, mgd	90	150	260
Calculated additional head loss due to diffuser[c] at design peak flow, in feet	1.9	2.1	6.1
Design friction factor in diffuser, f (Darcey-Wiesbach)	0.026	0.030 (legs) 0.026 (stem)	0.024
Manning roughness n, equivalent to f-values above	0.0155	0.0155	0.015

[a] Diffuser constructed by bulkheading end of outfall and drilling holes in main pipe. [b] Includes 2 slots in each end bulkhead, each slot being equivalent to one 9 in. diameter port. [c] Includes pipe friction in diffuser but not density head due to changes in depth.

In addition to the disappearance of the sewage bubbles, the sewage field or "sleek" has become so indistinct as to be indiscernible to other than a practiced eye. Density differences are reduced to the point that wave patterns move through the dilute sewage-sea water without discontinuity; reflective and refractive properties are so little changed as to make it almost impossible to see the sewage field.

Prior to the installation of diffusers, the area immediately over the points of discharge smelled faintly, but distinctly, of sewage and industrial wastes and, at times, the odor was wind-borne for as much as 1 mile. With diffusion, the problem of odors has ceased except during calms when a faint odor may be detected directly over the discharges. With even the slightest breeze this odor is so quickly dissipated as to be indiscernible a distance of 1,000 ft. The vicinity of the sewer outlets has, for many years, been a favorite fishing grounds for a number of commercial sports-fishing boats that depend on the attraction of the sewage effluent for an assurance of a fair day's catch of bottom fish. Since the use of diffusers, these fishing craft have been noted to station themselves at times directly over the diffusers.

Benthic studies in the vicinity of Whites Point disclose that ocean plant life has ceased for a distance of about 1/2 mile in all directions from the termini of the diffusers, possibly caused by toxic effects of industrial wastes, lessened salinity, or a decrease in transmitted sunlight (necessary for plant growth) resulting from increased turbidity in the immediate vicinity of the sewage discharge. Since the operation of the diffusers, the surface turbidity immediately over the diffusers has decreased; on occasion, the turbidity has actually been less than that of the surrounding ocean waters. The explanation for this apparent anomaly seems to lie in the fact that much of the diluting water is drawn from the bottom layers of ocean water; turbidity from sewage effluent, added to the very clear bottom waters, apparently results in a less turbid mixture than that of the surface waters which usually contain plankton.

The effect on the concentration of coliforms in shore waters is not so easily demonstrated as is the effect on ocean water in the immediate vicinity of the point of discharge. Of advantage is the fact that submerged sewage fields generated by the diffusers are subjected to shoreward currents which are weaker than those at the surface, allowing more time for natural reduction of coliform concentrations. Additionally, much higher initial dilutions have been achieved with diffusers, although this advantage is partially lost because the dilution rate occurring in dilute sewage fields formed by diffusers is slower than that occurring in the narrow field originating from an open pipe. For these reasons the benefits to be derived from diffusion will depend on a great many factors other than the simple relationship between the sewage concentrations in diffused or undiffused fields.

The general pattern of improvement in density of coliforms has met expectations (Fig. 3). It is not possible to make direct comparisons between numerical results since many factors obscure the pattern of coliform distribution resulting from the discharge of sewage effluent through the diffusers: to wit, the continuing increase in discharged sewage effluent, overlapping effects of discharges from local systems, and the disturbance of the ocean floor with depth charges in the construction of outfalls.

Averaged data on the concentration of sewage effluent in the ocean immediately overlying the points of discharge have been compared in Table 2 for the periods prior to and subsequent to the use of each diffuser. The diffuser on the 90-in. diameter outfall has produced a sewage field averaging about 1 part sewage effluent to 400 parts of sea water during the winter-spring regime when the sewage field is at the surface; when the field is submerged surface samples taken directly over the diffuser yield negative results, both for chloride changes and for the presence of coliform bacteria; no measurements have as yet been taken of the dilution obtained in the submerged fields.

Summary.—The outfall system of the Los Angeles County Sanitation Districts has provided a case history spanning more than 20 yr of experience in the disposal of a primary sewage effluent to a favorable ocean environment. Since 1953, the installation and operation of deep-water multiport diffusers has provided comparative experience indicating major reductions in odor, discoloration, and turbidity, all being significant improvements in the aesthetic acceptability of the discharge, as well as measurable reductions in bacterial populations. Inspection and maintenance of the diffusers have been accomplished in depths to 212 ft without difficulty.

DILUTION IN A RISING JET OF SEWAGE

To analyze horizontal discharge from an ocean outfall, assume that sewage effluent is discharged near the ocean floor from the end of a single pipe or port. Most important are determination of the rate of dilution of the sewage as it rises toward the surface of the salt water and the shape, density, and other characteristics of the jet or "rising column" of sewage. Discharge from a vertical or an inclined port or outlet will not be discussed herein because under such circumstances the effluent is given vertical momentum initially, thus rising to the surface faster and in a shorter path without other compensating influences.

TABLE 2.—IMPROVEMENT IN INITIAL DILUTION BY USE OF DIFFUSERS AT WHITES POINT

Outfall	Dilution[a]	
	Without diffuser	With diffuser
60-inch	40	170
72-inch[b]	30	300
90-inch	. . .[c]	400

[a] Based on average of maximum sewage concentrations at ocean surface over point of discharge, determined by ammonia concentration measurements. [b] 72-in. outfall was extended from depth of 110 ft to 155 ft at time diffuser was constructed. [c] Never operated without diffuser.

Except as indicated the ocean current is assumed to be nil, and ocean salinity and temperature are assumed uniform.

Description of the Problem.—When the sewage flows into sea water from the end of a pipe, it is immediately subjected to a buoyant force proportional to the difference in density between the sewage and the surrounding sea water. This force deflects the sewage jet, or stream, toward the surface and accelerates the sewage upward. The relative motion between the sewage jet and the sea water develops shear stresses. Turbulence is generated and mixing takes place first around the periphery of the jet or rising column and finally throughout the whole column. As the mixing progresses, density differences are decreased and the vertical force driving the sewage effluent toward the surface gradually decreases. Since the mixing is greatest around the periphery, that volume is decelerated most rapidly, and velocity gradients are developed across the column causing shear and turbulent diffusion throughout the entire column. From energy considerations, it can be shown that the upward velocity of the column as a whole gradually decreases as it rises, except possibly for a short distance at the start of its rise where differential densities are sufficient to accelerate the motion of the sewage effluent.

There is a complex interrelationship between the buoyant gravity force and the turbulent shear force. Both forces depend on the nature of the flow pattern, yet the flow pattern is determined by the forces acting; added to this dilemma are the uncertainties of the turbulent mixing process.

The appearance of the rising column is shown in Fig. 5 which is a photograph of a small jet in a laboratory tank. It is not surprising that there should be some difficulty in a direct physical analysis. The flow is inherently fluctuating due to its turbulent nature. Where a column of rising sewage effluent breaks the surface of the ocean, the so-called "bubble" shifts back and forth at random, making it impossible to identify any fixed point as the center; diluted sewage effluent arrives at the surface in numerous small "gusts." Divers report that large shifting eddies comprise the periphery of the column. At best, only time averages of the actual phenomenon can be considered.

The flow from an ocean outfall is essentially that of a submerged jet with buoyancy, with the flow field limited by the water surface and the ocean bottom. For a submerged jet without density difference in an infinite or semi-infinite fluid field, there are no external forces, such as gravity, affecting the flow; the only force is the internal shear. Consequently, considering the whole region of flow, the total momentum flux of the flow is constant. On the other hand, when external gravity forces deflect the jet into a curvilinear path, the momentum flux is no longer constant, and there is no simple way in which the buoyant force can be fitted into the equations for momentum flux. Likewise, the elementary principles of continuity and conservation of energy cannot be applied to the rising column. The volume of upward flow is constantly increasing due to the dilution, and energy is dissipated in turbulence.

A comprehensive experimental study of the hydrodynamics of submerged jets with buoyancy is needed. Some outstanding studies of diffusion of submerged jets without gravity effects have been published; especially noteworthy is the work of M. L. Albertson, Y. B. Dai, R. A. Jensen, and Hunter Rouse, whose analysis[4] is based on reasonable theoretical hypothesis and experimentally determined constants. It may be possible to extend their analysis to the more general case with a gravity force. Also of interest is the ingenious theoretical solution by G. Abraham[5] for the case of vertical discharge with buoyancy in an infinite medium.

Without the benefit of comprehensive research, one recourse is to resort to model experiments and empirical curve fitting. A. M. Rawn and H. K. Palmer[6] did this for both horizontal and vertical jets. The empirical formula de-

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[4] "Diffusion of Submerged Jets," by M. L. Albertson, Y. B. Dai, R. A. Jensen, and Hunter Rouse, Transactions, ASCE, Vol. 115, 1950, p. 639.

[5] "Diffusion in Submerged Circular Vertical Turbulent Water Jets with Density Difference Between the Jet and the Surrounding Water," by G. Abraham, Tech. Rept. 138-2, Hydr. Engrg. Lab. Wave Research Projs., Univ. of Calif., Berkeley, August, 1959.

[6] "Predetermining the Extent of a Sewage Field in Sea Water," by A M Rawn and H. K. Palmer, Transactions, ASCE, Vol. 94, 1930, p. 1036.

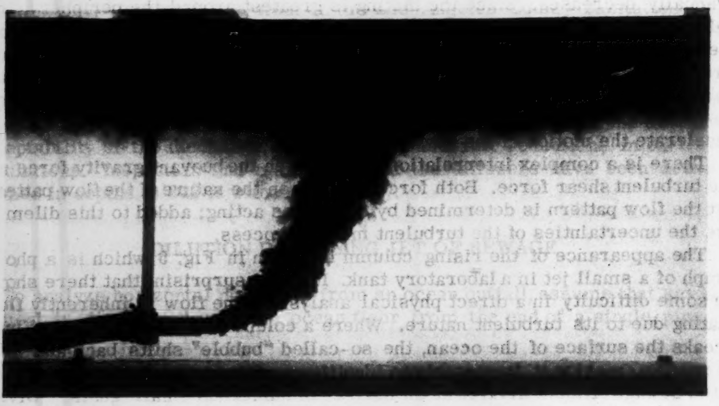

FIG. 5.—RISING COLUMN OF DYED FRESH WATER IN A SMALL LABORATORY TANK OF SALT WATER

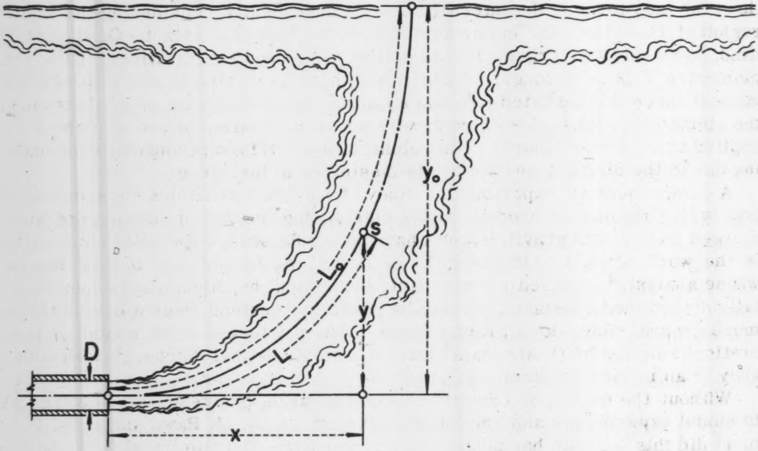

FIG. 6.—SKETCH OF SINGLE RISING COLUMN SHOWING NOMENCLATURE AND COORDINATE SYSTEM

rived from their experiments for dilution at the surface applies only to one particular set of units because the equation is dimensionally nonhomogeneous. With a somewhat different approach, utilizing dimensional analysis, it is possible to develop relationships in terms of dimensionless parameters. Furthermore, without considering some of the familiar dimensionless hydraulic numbers, questions arise as to the range of conditions over which the empirical formula is valid and as to the degree of similarity of flow patterns in model experiments to those of prototype cases. In addition, where dimensionless numbers are used, direct comparisons with other hydraulic phenomena are facilitated.

In the following four sections, the theory of hydraulic models is applied to the ocean outfall problem, and the original Rawn and Palmer[6] data for horizontal discharge are reanalyzed in this light. A new solution for the dilution is presented in graphical form.

Dimensional Analysis.—In this analysis, the dilution is defined as the ratio of the volume of the total mixture of sewage and sea water to the volume of the sewage fraction; in other words, $S = 1/p$, where p is the fraction of sewage in a sample. The dilution S is not necessarily the dilution at the ocean surface but is considered a variable depending on y, the height above the center of the outlet, as defined by the schematic diagram in Fig. 6. When $y = y_0$, the total depth from center of the outlet to the water surface, then $S = S_0$ the dilution at the top of the rising column. Furthermore, the dilution will also vary across the jet, becoming greater toward the periphery; hereinafter, S will denote the minimum dilution, which occurs along the axis of the jet. An additional complication is that turbulent fluctuations will cause the dilution at any one point to vary considerably with time, but since these fluctuations are not of especial importance, the analysis will deal only with the time-average dilutions.

In the simplified case with no temperature gradients or currents, the dilution S_0 in the center of the rising column at the surface may be considered a function of the following independent variables:

Symbol	Definition	Dimensions
y_0	Total depth from center of outlet to surface	L
D	Initial diameter of jet (same as diameter of pipe, port, or nozzle if there is no jet contraction)	L
V	Nominal jet velocity = $4Q/\pi D^2$, where Q = discharge	L/T
g'	Apparent acceleration due to gravity (Eq. 1)	L/T^2
ν	Kinematic viscosity of the sewage	L^2/T

The small difference between the kinematic viscosities of sewage and sea water is inconsequential; hence, only one viscosity is included in the dimensional analysis. Moreover, the mass densities of sewage and receiving water are considered the same as far as inertia of the fluids is concerned; the density difference is important only in the buoyancy calculations.

The apparent acceleration due to gravity, g', is the acceleration caused by the buoyant force, and is related to the ordinary gravitation constant, g, by the relation

$$g' = g \frac{\Delta s}{s} \quad \text{(1)}$$

in which s is the specific gravity of sewage and Δs denotes the difference in specific gravities of sea water and sewage.

Provided the discharge pipe is raised slightly above the ocean floor at the outlet end, as is usual, the actual distance from the center of the outlet to the bottom appears to be an unimportant variable in most practical cases, because the jet quickly turns upward away from the ocean bottom.

Including S_o, which is dimensionless, there are six pertinent variables, involving two fundamental units, length and time. Therefore, by the Pi Theorem, there are $6 - 2 = 4$ independent dimensionless numbers which can be formed from these six variables. It is most convenient to choose them as follows:

(a) S_o

(b) y_o/D

(c) $$\mathsf{F} = \frac{V}{\sqrt{g' D}} \text{ (Froude number)} \quad \ldots\ldots\ (2)$$

and

(d) $$\mathsf{R} = \frac{V D}{\nu} \text{ (Reynolds number)} \quad \ldots\ldots\ (3)$$

Thus S_o may be considered a function of the other three parameters,

$$S_o = f(y_o/D, \mathsf{F}, \mathsf{R}) \quad \ldots\ldots\ (4)$$

where F and R are the familiar Froude and Reynolds numbers in hydraulics.

The Reynolds number used is for the jet at its origin, and characterizes the relative importance of viscosity. For $\mathsf{R} > 2{,}000$, the jet flow will usually be turbulent, but the turbulence is probably not fully developed until R reaches 10,000 or 20,000. As R increases still further, the kinematic nature of the flow may be expected to change little.

The Froude number gives the relative influence of gravity on the jet. For a free liquid jet discharging horizontally into air,

$$\mathsf{F} = \frac{V}{\sqrt{g D}} \quad \ldots\ldots\ (5)$$

For small values of F, the jet is deflected sharply by gravity, whereas for large values of F, the jet has more momentum and the trajectory turns downward more gradually. For a submerged jet the Froude number, as defined in Eq. 2, has the same significance as it does for the case of the liquid jet into air, assuming here, as in the discharge into air, that friction is a secondary influence in determining the shape of the flow pattern near the point of discharge. For the submerged jet in the laboratory tank shown in Fig. 5, the Froude number $\mathsf{F} = 2.2$ and the depth to diameter ratio $y_o/D = 15.3$. This condition could correspond to a prototype discharge of 139 mdg from an 84-in. diameter outfall at a depth of 108 ft, using Froude-number similarity.

With dimensional analysis, the number of independent quantities has been reduced to three, each of which has a particular physical significance. However, the actual functional relationship between S and the three variables y_o/D, F, and R must be determined experimentally.

Hydraulic Similitude and the Use of an Hydraulic Model.—If the three parameters are identical for two distinct cases, then S_o must also be identical for the two cases. The fundamental requirement of any model study is to make the relevant dimensionless numbers the same in the model as in the prototype. Geometric similarity is certainly necessary, but far from sufficient. However, it is virtually impossible in any model study to achieve similarity for both Froude and Reynolds numbers simultaneously;[7] this is found to be the case in the ocean outfall problem under consideration.

For a submerged jet with no buoyancy, Albertson, et al[4] made the basic assumption that the turbulent flow pattern is dynamically similar for all Reynolds numbers, provided of course that the jet flow is fully turbulent. Their experimental results appear to justify the assumption. Since the rising column of sewage is a submerged jet with buoyancy, it is basically a combined diffusion and gravitational phenomenon, with the Reynolds number characterizing the turbulent diffusion and Froude number, the buoyancy effect. Therefore, it will also be assumed that the flow pattern for the rising column of sewage is independent of the Reynolds number in the fully turbulent range. To test this assumption, the original data of Rawn and Palmer[6] were re-examined to determine if any relationship between dilution and R could be detected.

Re-examination of Rawn-Palmer Data for Reynolds Number Effect.—The original Rawn-Palmer[6] study included 388 experimental observations of the dilution at the surface (S_o) for horizontal jets of fresh water into sea water. The nozzle diameter, D, ranged from 1/4 in. to 2 in. nominal diameter; various depths (y_o) up to 13.25 ft were used. (The experiments were made from a raft in Los Angeles Harbor, and thus simulated an ocean outfall situation, except that the discharge nozzle was not near the bottom as for a real outfall, but was cantilevered down from the raft. However, as previously indicated, this is not considered an important difference.) For each of these runs the values of y_o/D, F, and R were calculated from the original unpublished basic data. Careful analysis of these data by multiple plotting revealed no appreciable influence of Reynolds number which ranged from 5,000 to 40,000.

Reynolds numbers for actual outfalls may easily be of the order of 10^5 or 10^6. For example, a flow at 10 ft per sec from a 1-ft diameter port gives a Reynolds number of approximately 10^6. As the Reynolds number becomes large, its importance as a similarity number markedly decreases for many hydraulic phenomena. It seems reasonable, then, to hypothesize that if S_o is independent of R in the range of 5,000 to 40,000 (sometimes a rather critical range) it remains so for larger R-values as well.

Consequently, for the simple case of a rising column in a homogeneous ocean, S_o was considered a function of only two variables, namely

$$S_o = f(y_o/D, \mathsf{F}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Model studies may be performed on the basis of geometric similarity and Froude's law alone, provided that the flow pattern in the model is fully turbulent.

Graphical Solution for the Dilution.—A graphical solution for S_o was readily formulated, assuming that R is no longer an important variable. Smoothed curves were fitted to the data with the results shown in Figs. 7 and 8. Since both sets of curves represent the same relationship between the variables S_o, y_o/D, and F, values of S may be obtained from either figure.

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[7] Engineering Hydraulics, Edited by H. Rouse, Chapter II "Hydraulic Similitude" by J. E. Warnock, John Wiley and Sons, Inc., 1950.

With results presented in this manner, some interesting observations are possible. For any given horizontal outfall the y_0/D ratio is fixed, and F is directly proportional to V or Q. When y_0/D is about 40 it appears from Fig. 7 that as F increases from 2 up to 5 there is a progressive decrease in S_0. When F increases beyond 5, S_0 remains almost constant up to F = 10, and then starts to increase rapidly as F goes still higher.

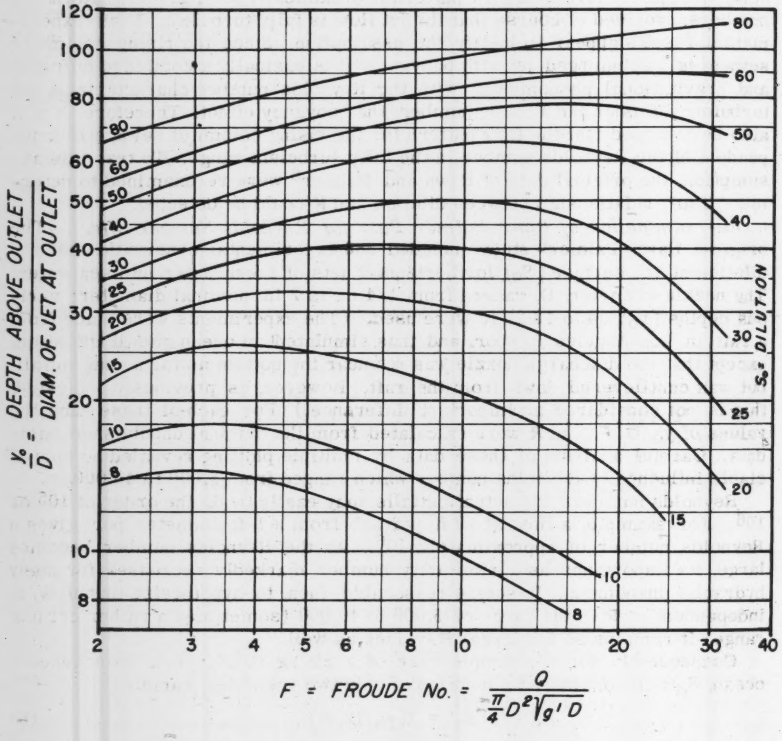

FIG. 7.—DILUTION AT TOP OF RISING COLUMN AS A FUNCTION OF y_0/D AND F FOR HORIZONTAL DISCHARGE (BASED ON RAWN-PALMER DATA)

When y_0/D is large (say around 80 to 100), then it takes relatively larger Froude numbers to produce the same effects on S_0 as those just described for $y_0/D = 40$. The depth becomes a more important factor than F. In other words, if the column must rise through 100 pipe diameters of depth, increasing the strength of the jetting action—and the horizontal displacement of the jet—for F-values up to 30 or 40, does not result in an increase in S_0 (or a downturn in the constant S_0 curves in Fig. 7).

The effect on S_0 of changing the several variables in Fig. 7 is shown by the straight lines in Fig. 9. If only the depth y_0 is increased keeping all other variables constant, then the change in dilution is found by following a vertical line. On the other hand, an increase in Q for a given outfall (fixed y_0 and D) is reflected only in an increase in **F** in direct proportion with no change in y_0/D. A further possibility is to decrease the size of the outlet, while maintaining the same depth and same discharge, thereby increasing V; since $\mathbf{F} \propto D^{-2.5}$, and $y_0/D \propto (D^{-1})$, the result is a line with slope 2/5 to 1 as shown. Another possibility is that a number of discharge outlets (N) are provided instead of only one, and that the outlet diameter D is decreased enough to leave the discharge velocity and aggregate discharge area constant. Since in this case $\mathbf{F} \propto D^{-0.5}$, the changed conditions follow the line with 2 to 1 slope.

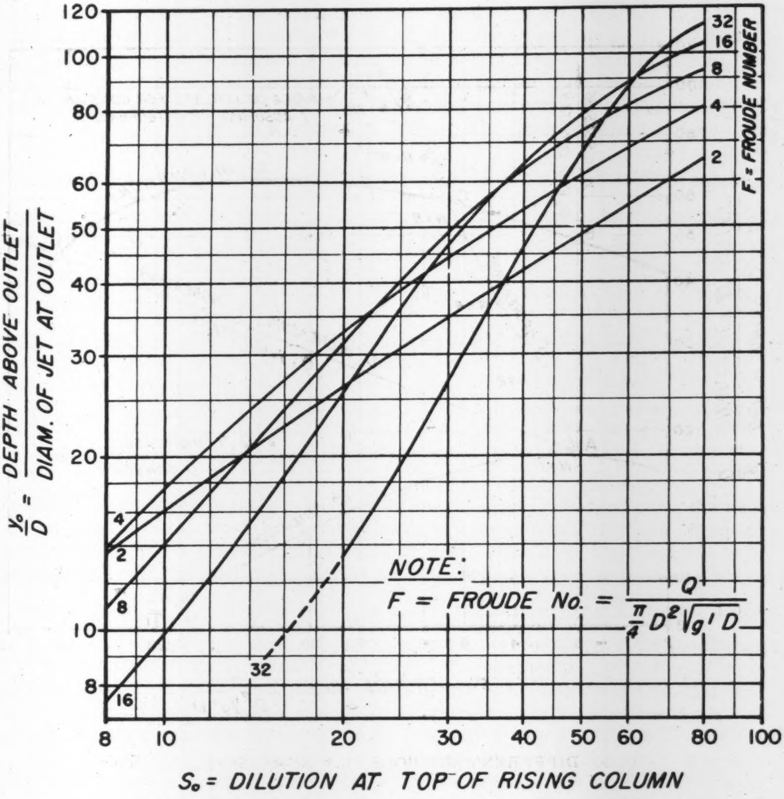

FIG. 8.—DILUTION AS A FUNCTION OF y_0/D FOR CONSTANT F FOR HORIZONTAL DISCHARGE (BASED ON RAWN-PALMER DATA)

Certain comparisons can be drawn from Fig. 9 to demonstrate the effectiveness of the several possible methods for increasing initial dilution of sewage effluent in sea water. For example, consider a discharge of 125 mgd through the open end of a 6-ft diameter outfall, lying at 108 ft of depth to pipe centerline; values for the dimensionless variables are $y_o/D = 18$ and $F = 3.0$. These are plotted as point A in Fig. 9. At that point either Fig. 7 or 9 shows that the dilution at the top of the rising column will be $S_o = 10$. Points B, C, and D in Fig. 9 illustrate three alternative methods for increasing S_o to 40. They are summarized in Table 3.

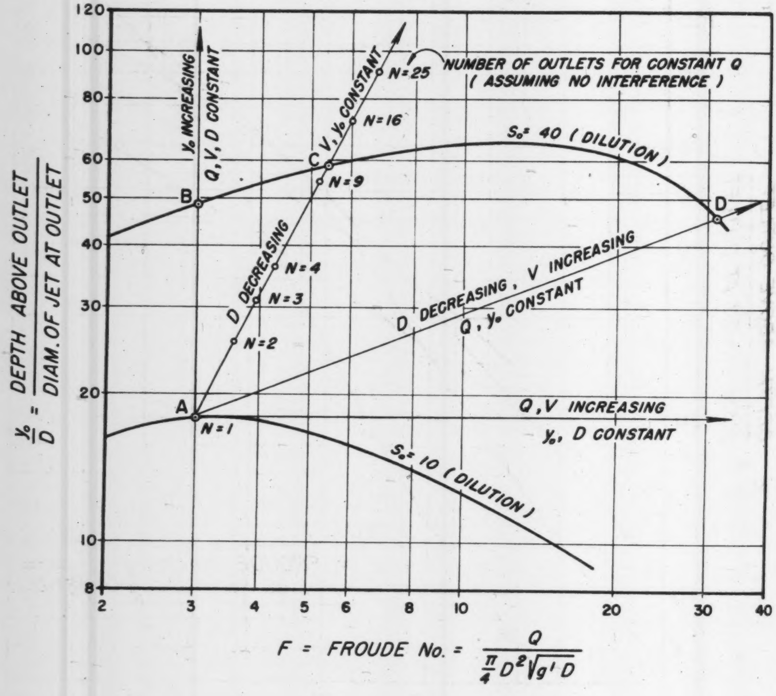

FIG. 9.—DIFFERENT METHODS FOR INCREASING DILUTION

The first possibility (point B) would involve extending the outfall offshore to a depth of 294 ft; unless increased distance from shore is needed for other reasons, this method of increasing the dilution is so costly as to be infeasible. The third possibility (point D) would probably be infeasible in most cases since increasing the velocity of discharge from 6.8 ft per sec to 44 ft per sec results in an increase in velocity head loss from 0.7 ft to 31 ft. The use of 10 or 11 separate discharge ports adequately spaced to avoid interference appears to be the most feasible alternative (point C). Little additional energy is required, and a diffusion structure can be built at a fraction of the cost required to greatly lengthen or deepen an outfall. Such a line of reasoning led to the decision by engineers of the Sanitation Districts to improve the ocean disposal facilities by means of multiple-outlet diffusers.

Other Investigations.—It has sometimes been suggested that the flow pattern surrounding a submerged ocean outfall is similar to convection over a heat source. Many investigations have been made of convection due to point or line sources of heat, one of the more recent such being a study by B. R. Morton, G. I. Taylor, and J. S. Turner.[8] When there is a source of heat in a fluid, the fluid adjacent to the source is heated, becomes lighter than the surroundings, and starts rising, thereby inducing convection currents. A rising column of fluid is formed over the heat source although no fluid has been injected into the system. The energy driving this type of flow is entirely derived from the heat or buoyancy. On the other hand, in the case of a submerged discharge from an ocean outfall, the energy available to the flow for entrainment and mixing is of two types: (1) potential energy due to the submergence of sewage in sea water $\left(\text{equivalent to } \frac{\Delta s}{s} y_o \text{ in hydraulic head}\right)$, and (2) kinetic energy of the jet. Consequently, the analogy is good only in case the kinetic energy of the discharge is small compared with the potential energy. The method of application of convection theory to the ocean outfall problem will be considered in detail in a later report, where density stratification of the receiving water will also be introduced as a variable.

TABLE 3.—EXAMPLES OF ALTERNATIVE METHODS OF INCREASING S_o

Point	y_o/D	y_o, in feet	D, in feet	Number of Outlets	Method
B	49	294	6	1	Increase depth of discharge to 294 ft.
C	58	108	1.86	10.4 (theor.)	Use 10.4 outlets (that is, 10 or 11) of D = 1.86 ft instead of one of D = 6 ft.
D	46	108	2.35	1	Increase discharge velocity by factor of 6.5, using D = 2.35 ft

Another possibility for analysis of a submerged jet with buoyancy is to assume that the dilution increases with L, the curvilinear length along the axis of the rising column (Fig. 6), at the same rate that dilutions in an ordinary submerged jet increase with distance along its axis. In effect, one is simply straightening out the rising column, and neglecting the effect of the difference in density. In essence, then, this approximation is the opposite of the one discussed above; the kinetic energy is now large compared with the potential energy due to density differences.

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[8] "Turbulent Gravitational Convection from Maintained and Instantaneous Sources," by B. R. Morton, G. I. Taylor, and J. S. Turner, Proceedings, Roy. Soc. A., Vol. 234, Jan-Mar., 1956, p. 1.

The second assumption has been used implicitly by E. K. Rice[9] (who discussed only vertical discharges) and by P. Cooley and S. L. Harris.[10] The latter conducted an outstanding experimental investigation under the direction of C. M. White on the use of submerged jets to prevent or destroy density stratification in a reservoir. Although they were dealing with relatively small density differences, and therefore high Froude numbers, the same principles apply.

Erman A. Pearson[11] has given a comprehensive summary of these and other investigations, and a list of references.

Interference between Jets.—Where there are many ports discharging from a diffuser, there will be some interference between the flow patterns established by the individual jets. If the diffuser ports are arranged in a long line, the flow of diluting water toward the diffuser will be essentially normal to the whole diffuser and thus not radial toward each individual rising plume of effluent. Such a restriction should tend to reduce the dilution somewhat below that which would be obtained from a single port.

Another type of interference is the actual contact of one rising column with another. Rawn and Palmer[6] observed that the diameter of the column at the top is approximately $L_o/3$, where L_o is the curvilinear length as shown in Fig. 6. Hence, when adjacent rising columns have travelled a distance L equal to about three times the port spacing, the columns may be expected to merge and further dilution will be substantially curtailed.

Stratification.—If the receiving water is stratified due to variations in temperature and/or salinity, the flow pattern may be considerably altered. In some instances where warm surface water overlies colder bottom water, a rising column of sewage may not reach the surface, as discussed previously. The sewage effluent discharged from a diffuser mixes with relatively large quantities of cold bottom water, sometimes permitting the mixture to become dense enough that it cannot penetrate a warm surface layer; thus, one of the important advantages of a diffuser may be the creation of a submerged sewage field, which might not occur with discharge from an open-ended outfall.

The problem of predicting possible submergence in the presence of density stratification is complex and cannot be discussed here except briefly. As a first approximation, one could assume an ocean consisting of only two distinct layers—a cold bottom layer and a warm surface layer. For the rise of the sewage in the cold water up to the level of the warm-water interface, the graphical relations in Figs. 7 and 8 can be used to determine an approximate value of the dilution S at that level. If s is the specific gravity of sewage and s_w and s_c the specific gravities of the warm and cold layers of sea water respectively, then the mixture will stay submerged below the warm surface layer if

$$\frac{(S - 1)\, s_c + s}{S} > s_w \quad \ldots\ldots\ldots\ldots\ldots (7)$$

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[9] "Discharge from Submerged Outfalls," by E. K. Rice, thesis presented to the Univ. of California, in Berkeley, Calif., in 1949, in partial fulfilment of the requirements for the degree of Master of Science.

[10] "The Prevention of Stratification in Reservoirs," by P. Cooley and S. L. Harris, Journal of the Institute of Water Engineers, London, Vol. 8, No. 7, November, 1954, p. 517.

[11] "An Investigation of the Efficacy of Submarine Outfall Disposal of Sewage and Sludge," by Erman A. Pearson, State Water Pollution Control Bd., Publication No. 14, Sacramento, Calif., 1956.

In addition, the density of the mixture should be enough greater than that of the overlying layer to overcome the residual inertia of the rising column.

The relation between dilution and temperature difference may be conveniently illustrated by an example, assuming the following data:

	Temperature, °F	Specific Gravity
Effluent	70	0.9987
Sea water (dilutent), salinity = 33,600 ppm	50	1.0258

Consider a column of sewage which rises from a depth of 150 ft to 80 ft, and in so doing becomes diluted with 50 parts of sea water at 50°F. Its specific gravity will then be about 1.0253, which is equivalent to plain sea water at 55° (same salinity, 33,600 ppm). If a thermocline exists above 80 ft, with water significantly warmer than 55°F, then a submerged sewage field may be anticipated. This is commonly the case in the summer and fall months along the southern California coast.

Even if the temperature gradients are not strong enough to prevent the sewage from surfacing, the sewage may still plunge under the surface after leaving the vicinity of the bubble, forming a field which is mostly submerged. This phenomenon was observed in the vicinity of the Whites Point outfalls before the construction of diffusers. Apparently the sewage was heavy enough to sink, but vertical momentum of the large plumes rising from the 5 and 6 ft open-ended pipes could not be overcome by the small density difference.

When the sewage field does stay at the surface, the temperature gradients are still beneficial in making the sewage field thicker, and hence more dilute. Since the edges of a rising jet become diluted most rapidly, this peripheral flow is stopped at a lower level even though the less dilute central core breaks through to the surface. Sampling in the vicinity of the diffuser for the 90-inch outfall at Whites Point indicates that the sewage field frequently extends 50 ft to 100 ft down from the surface when it is not submerged.

Ocean Currents.—Strictly speaking, all of the foregoing material applies to the case of no ocean current. While the aspirator action of the discharging jets from a diffuser is responsible for the intimate mixing of sewage and sea water, nonetheless the complete success of ocean disposal depends on ocean currents for gross exchanges of water. Without these currents the continuous discharge of sewage into the same ocean-water mass would gradually build up an intolerable concentration of sewage, and the sewage discharging from the diffuser would be remixed with older sewage.

If the currents are strong, the dilutions achieved in the ocean may considerably exceed those which may be calculated on the basis of single-port discharges into a calm ocean by Fig. 7. While the initial mixing results from the jet action, whether or not there is a current, the natural turbulence in the ocean may further increase the dilution if there is diluting water available. When a strong current sweeps past a diffuser, the large supply of clean water not only satisfies the demands of the jet entrainment process, but also provides sea water between the small individual diluted sewage streams generated by each port discharge. However, in this case these small individual fields soon mix with the intermediate clean water to form a rather large more or less homogeneous sewage field of a size commensurate with the size of the diffuser. It is this large field which, in the presence of a strong current, will be more dilute even though the initial dilution developed in the rising column itself may not be greatly different from that in a calm ocean.

Considering the total flow of water in a current stream, one may calculate by the continuity principle an average dilution which may be applied to the overall sewage field generated by a diffuser. If Q is the sewage discharge, U is the current velocity, h is the thickness of the sewage field generated, and b is the width of the stream intercepted (that is, the projection of the diffuser normal to the current) then the continuity equation simply states that

$$S_a Q = U b h \quad \text{(8)}$$

Unfortunately there appears to be no reliable way to predict the initial field thickness h in Eq. 8 other than by experience. Probably h will tend to decrease as the current velocity increases (and vice versa), thus somewhat offsetting the increase in S_a with increasing values of U. Nonetheless, unless the density stratification in the ocean is very strong, the field will gradually become thicker as it moves down-current as a result of vertical mixing along the bottom surface of the field and also along the top surface if the field is submerged. Hence an effective value of h can be taken at some distance downstream from the diffuser to allow for some vertical mixing. Values of 1/4 or 1/3 of the ocean depth are not considered unreasonable.

As an example of the use of Eq. 8, consider an onshore current of 0.8 ft per sec (0.5 knot) passing the diffuser for the 90-in. outfall at Whites Point (Fig. 2):

$$U = 0.8 \text{ ft per sec}$$

$$b = 2 \times 1200 \times \sin 63^\circ\, 30' = 2150 \text{ ft}$$

$$h = 60 \text{ ft (assumed, but preferably measured)}$$

$$Q = 180 \text{ mgd (average flow)} = 280 \text{ cfs}$$

$$S_a = \frac{0.8 \times 2150 \times 60}{280} = 370$$

It is important to note that this dilution is an average for the entire field, whereas the dilutions represented by Fig. 7 are minimum values, occurring at the center of the individual rising columns, before a large homogeneous field is formed downcurrent.

At this point one may well ask whether the dilution should be calculated in this way or by Fig. 7. If the average dilution for the field as a whole (S_a) is larger than S_o achieved in each rising column with no current, it probably indicates availability of more diluting water than that immediately taken up by the aspirator action. There will be local areas with higher than average concentration in the "bubbles," but as the current carries the field downstream, the natural turbulence in the ocean will cause internal mixing utilizing all the available diluting water and thus promoting uniformity of the field. Hence, in this case the effective dilution may be considered as the average S_a.

On the other hand, if S_o by Fig. 7 is larger than S_a by Eq. 8, then the current does not make additional water available over that drawn in by the initial jet mixing. The actual dilutions achieved over a diffuser will probably be less than S_o by Fig. 7 because of mutual interference between the rising columns (unless the port spacing is very large).

Apart from the beneficial effects of improving the initial dilution at the outfall site, the currents are of course an adverse factor when they carry the sewage field rapidly back to shore. An analysis of the behavior of sewage fields, once formed, is beyond the scope of the present paper, but the problems are briefly described in the following section.

Initial Dilution in Relation to Ultimate Disposal.—The discussion heretofore has dealt almost exclusively with the determination of the initial dilution of sewage with sea water at or near the point of discharge, whether there be single outlets or many. Although it is highly desirable to get as large an initial dilution as possible, that alone does not assure safe disposal. As indicated previously, it is also necessary that the site have favorable currents, and that the sewage effluent is discharged at a distance from shore sufficient to allow for further reduction of bacterial concentration before the diluted sewage reaches the shoreline.

That dilution alone cannot reduce the concentration of coliform bacteria to the California standard for salt-water bathing (10 per ml) is apparent in that the primary sewage effluent may have concentrations of the order of 10^6 per ml; reduction of concentration by five orders of magnitude is necessary from the time the sewage is discharged into the ocean until it reaches the surf zone in order that shoreline waters are not to exceed 10 coliforms per ml. It is doubtful if any reasonable diffusion system, assisted by further mixing in the ocean as the sewage field travels toward shore, could consistently produce physical dilutions of more than 1,000 to 1. Thus a reduction of bacterial count by two more orders of magnitude must come about through processes such as mortality and sedimentation in the ocean.

In the disposal process the engineer can control only the initial dilution through the choice of outfall site and type of diffuser. Removal of coliforms by natural processes is beyond his direct control; therefore a diffuser which produces a high initial dilution yields greater assurance of good results. Moreover, with a diffuser all other requirements for sewage disposal can be met including turbidity, toxicity, dissolved oxygen levels, odors, and slicks.

An analysis of what takes place in the sewage field after it leaves the vicinity of the diffuser depends on an understanding of the mechanism of turbulent diffusion in the ocean as well as sedimentation and mortality of bacteria. This paper is not broad enough in scope to discuss these factors, but other investigations[12,13,14,15,16] along these lines (primarily in connection with the design of a new ocean outfall for the city of Los Angeles in 1955-56) have been performed.

Summary.—The original Rawn and Palmer[6] experimental data for submerged jets with buoyancy have been reanalyzed to produce Figs. 7 and 8 which give the dilution in the center of rising column of sewage at the ocean surface

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[12] "Studies on Coliform Bacteria Discharged from the Hyperion Outfall," by S. C. Rittenberg, Final Bacteriological Report to Hyperion Engineers by Allan Hancock Foundation, Univ. of So. Calif., Los Angeles, Calif.

[13] "Sewage Disposal in Santa Monica Bay, California," by C. G. Gunnerson, Proceedings, ASCE, Vol. 84, No. SA1, February, 1958.

[14] "Ocean Outfall Design," Hyperion Engineers (a joint venture), Los Angeles, October 15, 1957.

[15] "Methods of Analysis of the Performance of Ocean Outfall Diffusers with Application to the Proposed Hyperion Outfall," by N. H. Brooks, Report to Hyperion Engineers, Pasadena, Calif., April 5, 1956.

[16] "Diffusion of Sewage Effluent in an Ocean Current," by N. H. Brooks, Proceedings, First Internatl. Conf. on Waste Disposal in the Marine Environment, Univ. of Calif., Berkeley, Calif., July 23, 1959.

as a function of two dimensionless numbers: the ratio of depth to jet diameter and the Froude number for the discharge. This new relationship is considered to have more reliability than the earlier empirical equations, inasmuch as the principles of hydraulic models and dimensional analysis have been utilized in developing Figs. 7 and 8.

The most practical way to achieve an increase in initial dilution appears to be to increase the number of discharge jets by means of a relatively long manifold or diffuser. If the receiving water is stably stratified, such as when warm water overlies cold, the small jets of sewage from a diffuser may never appear at the surface, a most desirable objective.

HYDRAULIC DESIGN OF DIFFUSERS

For a given ocean outfall, improvement of dispersal of sewage effluent is accomplished by use of outlet diffusion at the end of the outfall sewer. If the sewage is discharged at a single port or "en masse," its dispersion and dilution will be slower than if it is discharged over a large area through a number of ports. In fact, without the use of multiple-outlet diffusers, other conditions being equal, much longer outfalls into deeper water are necessary to provide the same degree of dispersion and consequent shore protection.

An effective and simple type of diffuser is one which distributes the outflow through many ports over a large area with minimum head loss and interference between rising columns. Diffusers which discharge several radial jets from a single small manifold chamber have been used, but are not as effective because of the interference between rising columns. The following material presumes a diffuser consisting of one long pipe, or several branching ones, with discharge ports at intervals along the pipes.

Basic Hydraulic Requirements.—The successful design of a long diffuser with a large number of outlet ports requires careful hydraulic calculations. The basic hydraulic requirements of such a diffuser are outlined herewith.

Flow Distribution.—The division of the outflow between the various ports should be fairly uniform. If the diffuser is laid on a sloping sea bottom, it will be impossible to achieve uniform distribution between ports for all rates of flow. In such cases, it is advisable to make the distribution fairly uniform at low or medium flow, and let the deeper ports discharge more than the average port discharge during high rates of flow. To allow substantially less than average discharge from deeper ports is considered unsafe from the point of view of possible clogging of the deeper part of the diffuser.

Velocity in Diffuser.—The flow velocity in all parts of the diffuser should be high enough to prevent gross deposition of sludges or grease. For settled sewage, velocities of 2 fps to 3 fps at peak flow are adequate (but borderline) since these will tend to scour material settled during low flow periods. If deposition takes place in any part of the diffuser over an extended period of time, the cross section of the pipe or outlet may become so constricted that locally the velocity will be reduced, a cycle that would accelerate the deposition process. The final result may be complete clogging of the terminal ports and failure of the diffuser to completely perform its dispersal function.

Ease in Cleaning.—Even carefully designed diffusers will require occasional cleaning to remove accumulated grease, slimes, and grit as cited previously under the heading "Outfall System of the Los Angeles County Sanitation Districts: Inspection and Maintenance of Diffusers." These accumulations tend to increase the apparent friction factor (mainly by decreasing cross-sectional area), thereby reducing flow from offshore ports and increasing flow from inshore ports. Cleaning can be accomplished by flushing or pulling a ball through the line.

Prevention of Sea Water Intrusion.—All ports should flow full in order to prevent the intrusion of sea water into the pipe. Sea water entering the pipe will be stagnant and will tend to trap grit and other settleable matter. Such deposits reduce the hydraulic capacity of the diffuser, thereby limiting its usefulness for future years when higher flows might be expected.

Total Head Loss.—If effluent pumping is necessary or the available gravity head is limited, the total head loss in any proposed diffuser should be kept reasonably small.

Port Design.—The outlet ports may quite satisfactorily be circular holes in the side of the pipe without nozzles or tubes or other projecting fittings. For optimum dilution the jets should discharge horizontally, with no initial upward component of velocity. The inside of the hole should preferably be bellmouthed to minimize clogging and to provide a discharge coefficient which will remain constant over a period of years.

Hydraulic Analysis.—The hydraulic analysis of a multi-port diffuser is basically a problem in manifold flow and is somewhat complex. A discourse on the subject is not within the purview of this paper, but may be found elsewhere.[17,18] The purpose of the following analysis is primarily to illustrate one manner in which a diffuser can be designed, and to demonstrate some basic principles peculiar to the design of ocean outfall diffusers.

Gravity Effects.—Several gravity effects are important in diffuser flow. The effects of density difference on the rising column has previously been discussed under the heading "Dilution in a Rising Jet of Sewage: Dimensional Analysis" where it was stated that the shape of the discharge jet near the port is governed only by the Froude number F as defined by Eq. 2, assuming ideal flow conditions upstream of the port. According to Rouse,[19] for a circular orifice in a large tank, the Froude number should be greater than 0.59 in order for the orifice to flow full. For a rounded port, it is reasonable to take $\mathsf{F} > 1$ as the criterion for flowing full. With every port in the diffuser flowing full, there is no way in which the sea water may re-enter the pipe, once initially expelled, and the diffuser will continue to remain full of sewage effluent.

In making hydraulic calculations, the pertinent pressure at any point in the diffuser is the pressure differential between the fluid inside the diffuser and the sea water outside at the level of the port. Working in reverse order from the deepest or farthest point of a diffuser backward, the decrease in depth tends to increase the pressure differential between the sewage inside the diffuser and the sea water outside, in spite of the fact that the pressure of both sewage and sea water decreases. Henceforth, the use of the terms "pressure" and "pressure head" will herein refer to the pressure differential. The change

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[17] "Mechanics of Manifold Flow," by J. S. McNown, Transactions, ASCE, Vol. 119, 1954, p. 1111.

[18] "Application of Conformal Mapping to Divided Flow," by J. S. McNown and E. Y. Hus, Proceedings of the First Midwestern Conf. on Fluid Dynamics, J. W. Edwards, Ann Arbor, Mich., 1951, p. 143.

[19] "Elementary Mechanics of Fluids," by H. Rouse, John Wiley and Sons, Inc., 1946, p. 105.

of this pressure head due to a change of elevation of Δz will be equal to $\frac{\Delta s}{s}\Delta z$.

To the hydraulic engineer designing sewage diffusers for the first time, everything will seem upside down. Indeed, it is quite feasible to visualize the diffuser flow pattern as being inverted, with the vertical scale reduced by the factor $\Delta s/s$ and with the diffuser discharging water into air with the same Froude numbers. The analogy should also help to make clear why it is sometimes difficult to achieve uniformity of discharge from a diffuser built on a sloping ocean floor.

Characteristics of Flow from a Single Port.—The hydraulic analysis of a diffuser is essentially a step-wise process starting at the extreme outer end. The ports are assumed to be far enough apart so that the flow in the vicinity of any one port is independent of the rest of the diffuser flow. This is a reasonable assumption, as the port spacing in an adequate diffuser would be at least 10 port-diameters, and probably considerably more. The discharge from each port is figured separately in turn, and added to the quantity of flow carried by the diffuser pipe downstream. Between consecutive ports, the effective pressure head is increased by the amount of the friction loss and the density head $\left(\frac{\Delta s}{s}\Delta z\right)$. The key to the problem is the analysis of lateral discharge from a port in the side of a pipe.

The rate of discharge, q, from an orifice or port in the side of a pipe is expressed by

$$q = C_D a\sqrt{2\,g\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

in which C_D is the discharge coefficient, a denotes the cross-sectional area of port, and E is the total head in the main flow at the port. The total head, E, includes the pressure head in the main pipe relative to the ocean at the location of the port plus the velocity head of the main flow. In the neighborhood of the discharge port, it is assumed that there is no energy loss for the main flow in passing the port. In other words, there is perfect pressure recovery compensating for reduction in velocity head in the main flow because of the lateral discharge. McNown[17] has shown this to be a good assumption.

The discharge coefficient, C_D, is not a constant along the diffuser, but decreases as the velocity head $(V^2/2\,g)$ becomes a larger part of the total energy (E). By an extension of a theoretical analysis of branching flow by McNown and Hsu[18] it has been found that C_D can be expressed as a function of the ratio $\frac{V^2}{2\,g}/E$ as shown in Fig. 10. Theoretically the curve in Fig. 10 applies only to small discharges from small holes with diameter less than one-fourth of the main pipe diameters, as is always the case with diffusers. For small q, the velocities upstream and downstream of the port are approximately equal (that is, $V_n \approx V_{n-1}$) and either V_n or V_{n-1} may be used to calculate the ratio $\frac{V^2}{2\,g}/E$, although V_{n-1} is the more convenient. Furthermore, it is presumed that the discharge coefficient for negligible velocity of approach $\left(\frac{V^2}{2\,g}/E \rightarrow 0\right)$ is 0.61 for a sharp-edged port and 0.91 for a rounded port. These values are reasonable for discharge from a tank or reservoir.

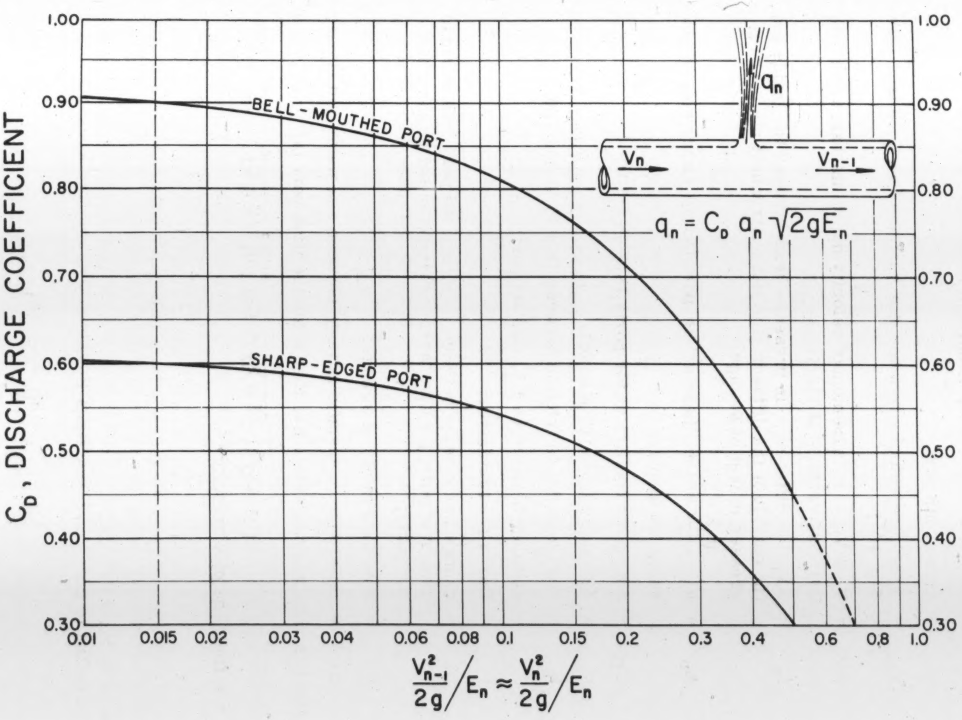

FIG. 10.—DISCHARGE COEFFICIENT FOR A SMALL PORT ON THE SIDE OF A PIPE

Calculation Procedure.—The whole calculation procedure as used in the design of a diffuser may be formulated mathematically as follows:

Let: D = diameter of pipe;

d_n = diameter of nth port, counting n from offshore end;

a_n = area of nth port;

V_n = mean pipe velocity between nth port and $(n + 1)^{th}$ port (see Fig. 10);

ΔV_n = $V_n - V_{n-1}$ = increment of velocity due to discharge from nth port (or group of ports);

h_n = $\Delta P_n/\gamma$ = difference in pressure head between the inside and the outside of the diffuser pipe just upstream of nth port (expressed in feet of sewage);

E_n = $h_n + \frac{V_n^2}{2\,g}$ = total head at nth port (same either side by assumption above);

C_D = discharge coefficient for ports (see Fig. 10);

q_n = discharge from the nth port;

h_{fn} = head loss due to friction between $(n + 1)$ and nth port;

L_n = distance between $(n + 1)$ and nth port;

f = Darcy friction factor;

Δz_n = change in elevation between $(n+1)$ and nth port (measured to center of port; positive when $(n+1)$ port is not as deep as the nth port);

s = specific gravity of sewage;

Δs = difference in specific gravity between sea water and sewage

First it is necessary to select E_1; then q for the first port is:

$$q_1 = C_D\, a_1 \sqrt{2\,g\,E_1} = C_D \frac{\pi}{4} d_1^2 \sqrt{2\,g\,E_1} \qquad (10)$$

Next, one finds the pipe velocity

$$V_1 = \Delta V_1 = \frac{q_1}{\frac{\pi}{4} D^2} \qquad (11)$$

and velocity head $V_1^2/2\,g$.

Proceeding to port No. 2, one finds E_2 by

$$E_z = E_1 + h_{f1} + \frac{\Delta s}{s} \Delta z_1 \qquad (12)$$

The ratio $\frac{V_1^2}{2\,g}/E_2$ is calculated for use in Fig. 10 to find C_D. Then

$$q_2 = C_D\, a_2 \sqrt{2\,g\,E_2} \qquad (13)$$

and

$$V_2 = V_1 + \Delta V_2 = V_1 + \frac{q_2}{\frac{\pi}{4} D^2} \qquad (14)$$

This procedure is continued step by step back up the diffuser using the general relations:

$$q_n = C_D a_n \sqrt{2 g E_n} \qquad (15)$$

$$C_D = \phi\left(\frac{V_{n-1}^2}{2 g} \Big/ E_n\right) \qquad (16)$$

$$\Delta V_n = \frac{q_n}{\frac{\pi}{4} D^2} \qquad (17)$$

$$V_n = V_{n-1} + \Delta V_n \qquad (18)$$

$$E_{n+1} = E_n + h_{fn} + \frac{\Delta s}{s} \Delta z_n \qquad (19)$$

and

$$h_{fn} = f \frac{L_n}{D} \frac{V_n^2}{2 g} \qquad (20)$$

A tabular form was devised to facilitate the calculations as illustrated by Table 4. If the port discharges and pipe velocities change slowly, it is expedient to make the stepwise calculations for small groups of ports, as was done in Table 4. In this case, Eq. 17 is changed to read

$$\Delta V_n = m \frac{q_n}{\frac{\pi}{4} D^2} \qquad (21)$$

wherein m is the number of ports considered in a group.

By the nature of the calculations, it is apparent that one cannot decide on a particular total flow before starting the calculations. It is necessary to estimate the flow from the end port (q_1) which will correspond to the desired total flow.

Selection of Port Sizes and Spacing and Pipe Sizes.—During the process of the calculation, the designer is at liberty to change the pipe size, the port size, and/or the port spacing. To keep the velocity high enough at the end of the diffuser, it is sometimes necessary to reduce the size of the pipe in one or more steps from beginning to end of the diffuser. The size of the discharge ports may be varied in order to keep the discharge uniform from port to port. The spacing between ports is rather inflexible, inasmuch as practical considerations dictate that the spacing be either equivalent to the length of a pipe section or multiple or simple fraction thereof. The entire design process inevitably requires some trial and error arrangements in order to get one arrangement which is satisfactory at various total rates of flow.

For a diffuser which is laid on a zero slope, the relative distribution of flow would be the same at all rates of discharge. This is because all the head terms are proportional to the square of the velocity. In that case, where there are no differential elevations, one calculation would suffice for all rates of flow. For example, to double the rate of flow, one would need only to quadruple all the heads and double all the velocities and discharges.

It is essential that the end of the diffuser pipes be bulkheaded, otherwise the flow will not be forced out of the discharge ports near the end of the diffuser, and an excess of flow will be discharged through the open-ended pipe. The bulkheads should be removable for flushing the line.

In the process of making the hydraulic calculations it was found that a good rule of thumb was to assure that the sum of all the port areas is less than the cross-sectional area of the outfall pipe. In fact, it was quite apparent that it is impossible to make a diffuser flow full if the aggregate jet areas exceeds the pipe cross-section area, since that would mean that the average velocity of discharge would have to be less than the velocity of flow in the pipe and that the flow would have to be decelerated before being discharged from the ports. This is not physically possible; consequently some of the ports would not flow full, or not flow at all, and the diffuser would not work properly. It is probable that the principal reason for the difficulties of some of the early multiple-outlet diffusers, such as the Deer Island Outfall[4] in Boston, Mass. was the failure to observe that simple criterion.

Application to Outfalls of the Los Angeles County Sanitation Districts.—The foregoing calculation procedures is illustrated by a set of design calculations for the diffuser for the 90-in. outfall (see Fig. 2). Table 4 lists hydraulic calculations for a low flow of 79 mgd, a condition selected as an example because the density head terms $\frac{\Delta s}{s} \Delta z_n$ have a significant effect on the results. In the following material, the several steps in the hydraulic design of the diffuser for the 90-in. outfall will be reviewed.

Initially, it was necessary to select the location, orientation, and length of the two diffuser legs from oceanographic and economic considerations, including initial dilutions obtainable, prevailing currents, temperature profiles, and costs of alternatives. It was decided to make two diffuser legs 1,200 ft long with an included angle of 127°, the bisector of the angle being approximately normal to the coastline. In this way, the width of the band of current intercepted (or the width of sewage field formed) was largest for a shoreward current, and least for a longshore current. No ports were provided in the main trunk, as was done in the 72-in. outfall, since the width of the sewage field would thereby only be increased for longshore currents which are not critical. Furthermore, the grade of the pipe is much steeper on the main trunk making a balanced hydraulic design more difficult to achieve if ports were to be placed in the main pipe.

The two diffuser legs consist of 60-in. reinforced concrete pipe with one port in each 24-ft section, facing alternate directions. The two legs were laid out with as nearly identical profiles as possible, to ensure symmetry and equal distribution of the flow.

From the detailed hydraulic calculations, the designer is able to select reasonable port and pipe diameters. In the case in question it was found that the diffuser pipes could satisfactorily be of a single diameter throughout—60 in.

TABLE 4. —SAMPLE HYDRAULIC CALCULATIONS FOR DIFFUSER DESIGN, 90 - IN. OUTFALL DIFFUSER, FLOW OF 79 MGD

Port Number (1)	Distance from End,[a] in Feet (2)	Port Diameter, d_n, in Inches (3)	Port Area, a_n, in Square Feet (4)	Pipe Diameter, D, in Feet (5)	Total Head, E_n, in Feet, From Col. 18 (6)	Ratio $\frac{V_{n-1}^2}{2g}$, Col. 12 ÷ Col. 6 (7)	Discharge Coefficient, C_D, from Fig. 10 (8)	Port Discharge, in cfs, q_n = Col. 8 x Col. 4 X $\sqrt{2g \text{ Col. } 6}$ (9)	Increment in Pipe Velocity, in fps, $\Delta V = \frac{m}{A}$ (Col. 9) (10)	Pipe Velocity, V_n, in fps (11)	Velocity Head, $V_n^2/2g$ in Feet (12)	Friction Factor, f (13)	Distance to Next Port, L_n, in Feet (14)	Head Loss, in Feet, h_{fn} = Col. 12 x Col. 13 x Col. 14 ÷ Col. 5 (15)	Decrease in Depth, Δz_n, in Feet (16)	Density Head, $\frac{\Delta s}{s} \Delta z_n$, in Feet (17)	Total Head, in Feet, E_n = Col. 6 + Col. 15 + Col. 17 (18)	Remarks $\Delta s/s = 0.026$, $A = \pi D^2/4 = 19.63$ sq ft, m = number of ports in group in Col. 1 (19)
1	0	15.0	1.227	5.00	0.199	0	0.91	4.00	0.204	0.20	0.00065	0.024	24	0.000+	0.43	0.011	0.210	Slope of pipe = 1.8% for first
2-3	24	7.5	0.307	"	0.210	0.003	0.91	1.03	0.105	0.31	0.0015	"	48	0.000+	0.86	0.022	0.232	504 ft
4-5	72	"	"	"	0.232	0.006	0.91	1.08	0.110	0.42	0.0027	"	"	0.001	"	"	0.255	f = 0.024 is equivalent to
6-7	120	"	"	"	0.255	0.011	0.905	1.13	0.115	0.53	0.0044	"	"	0.001	"	"	0.278	Manning n = 0.015
8-9	168	"	"	"	0.278	0.016	0.90	1.17	0.119	0.65	0.0066	"	"	0.002	"	"	0.302	
10-11	216	"	"	"	0.302	0.022	0.89	1.21	0.123	0.77	0.0093	"	"	0.002	"	"	0.326	
12-13	264	"	"	"	0.326	0.029	0.885	1.25	0.127	0.90	0.013	"	"	0.003	"	"	0.351	
14-15	312	"	"	"	0.351	0.036	0.875	1.28	0.130	1.03	0.016	"	"	0.004	"	"	0.377	
16-17	360	"	"	"	0.377	0.043	0.87	1.32	0.134	1.16	0.021	"	"	0.005	"	"	0.404	
18-19	408	6.5	0.230	"	0.404	0.052	0.86	1.01	0.103	1.26	0.025	"	"	0.006	"	"	0.432	
20-21	456	"	"	"	0.432	0.057	0.855	1.04	0.106	1.37	0.029	"	"	0.007	"	"	0.461	
22-23	504	"	"	"	0.461	0.063	0.85	1.07	0.109	1.48	0.034	"	"	0.008	0.34	0.009	0.478	Slope of pipe = 0.7% for
24-25	552	"	"	"	0.478	0.071	0.84	1.07	0.109	1.59	0.039	"	"	0.009	"	"	0.496	remaining 696 ft.
26-27	600	"	"	"	0.496	0.079	0.835	1.08	0.110	1.70	0.045	"	"	0.010	"	"	0.515	
28-29	648	"	"	"	0.515	0.087	0.825	1.09	0.111	1.81	0.051	"	"	0.012	"	"	0.536	
30-31	696	"	"	"	0.536	0.095	0.82	1.11	0.113	1.92	0.057	"	"	0.013	"	"	0.558	
32-33	744	"	"	"	0.558	0.102	0.81	1.12	0.114	2.03	0.064	"	"	0.015	"	"	0.582	
34-35	792	"	"	"	0.582	0.110	0.80	1.13	0.115	2.15	0.072	"	"	0.017	"	"	0.608	
36-37	840	"	"	"	0.608	0.118	0.795	1.14	0.116	2.27	0.080	"	"	0.018	"	"	0.635	
38-39	888	"	"	"	0.635	0.126	0.785	1.16	0.118	2.39	0.089	"	"	0.020	"	"	0.664	
40-41	936	"	"	"	0.664	0.134	0.775	1.17	0.119	2.51	0.098	"	"	0.022	"	"	0.695	
42-43	984	"	"	"	0.695	0.141	0.77	1.19	0.121	2.63	0.107	"	"	0.025	"	"	0.729	
44-45	1032	"	"	"	0.729	0.147	0.765	1.21	0.123	2.75	0.117	"	"	0.027	"	"	0.765	
46-47	1080	"	"	"	0.765	0.153	0.755	1.22	0.124	2.87	0.128	"	"	0.029	"	"	0.803	
48-49	1128	"	"	"	0.803	0.159	0.75	1.24	0.127	3.00	0.140	"	"	0.032	"	"	0.844	
50-51	1176	"	"	"	0.844	0.166	0.745	1.27	0.129	3.13	0.152	"	24	0.018	0.17	0.004	0.866	Wye is 1200 ft. from end
Sum			13.95					Q=61.5 each log					1200	0.31	14	0.36		Q_{TOTAL} = 123cfs = 79 mgd

[a] When a group of ports is used, distances are to the centroid of the group.

[b] Aggregate port area for both legs is 27.9 ft.² or 63% of cross section of 90-inch outfall pipe.

After some trial and error calculations, a port size of 7.5 in. was chosen for the outermost 14 sections, and 6.5 in. for the other 34 sections in each leg. Quadruple-sized ports (15-in. diameter) were placed in the center of each of the end bulkheads in order to maintain favorable pipe velocity and to make use of the better than average diffusion opportunity at the extreme borders of the sewage field formed by the diffuser.

In Table 4, the Δz_n-values are based on an average measured grade of 1.8% for the outermost 500 ft and 0.7% for the inner 700 ft, thus accounting for a total drop of about 14 ft. With a 2.6% density difference, this drop makes a total head differential of 0.36 ft of fresh water head; that is, even under hydrostatic conditions the pressure head difference between inside and outside of the pipe would be 0.36 ft greater at the wye than at the end. From the top of Col. 6 and the bottom of Col. 18 in Table 4 it may be noted that the total head increases from 0.20 ft to 0.87 ft as one works backward from the ~~end of the~~ wye. Of this increase of 0.67 ft, 0.36 ft is due to change in elevation and only 0.31 ft to friction. Consequently, this set of calculations demonstrates the importance of considering the differential elevation at low flow. At a flow of 240 mgd, the computed total head was found to rise from 3.25 ft to 6.67 ft, an increase of 3.42 ft; of this amount 3.06 ft is for friction loss, with only 0.36 ft for change in elevation, as before.

The effect of the sloping bottom on the distribution of flow between the ports is apparent in Fig. 11 on which are plotted the distributions for a high flow of 240 mgd and a low flow of 79 mgd. The port sizes have been selected in a way which makes the port discharges practically uniform at the low flow, but at high flow, the offshore ports discharge on the average about 25% more than the inshore ports. Going shoreward from the end of the diffuser (that is, in the order in which the calculations are made), the flow from each port gradually increases for low flow primarily because of the increase in elevation. On the other hand, at high flow, the situation is reversed; in spite of the increasing elevation and accumulated pipe friction losses, the port discharge decreases very gradually because the discharge coefficient decreases faster than the head increases. It is seen that there are four factors which tend to change the port discharge: change in elevation, pipe friction, change in C_D, and change in port diameter. For a successful diffuser these four factors should be kept as nearly in balance as possible.

As indicated previously, an hydraulic design yielding discharge distributions such as those in Fig. 11 is safe inasmuch as there is virtually no danger of gradual choking of the offshore end of the diffuser. If the friction factor f became considerably higher than the design value of 0.024 (which is equivalent to Manning n = 0.015), the discharges from offshore ports would be decreased while the discharges from nearer-shore ports would be increased. Consequently, as the diffuser is designed, an increase in f would only tend to make the distribution of discharge more uniform at high rates of flow.

The velocity distribution within the diffuser pipe may be read from Table 4, Col. 11. At low flow (79 mgd), the velocity in the pipe has a minimum of 0.20 fps at the very end, and maximum of 3.12 fps just downstream from the wye. At the high flow (240 mgd), the corresponding velocity figures are 0.82 and 9.46 fps, respectively. Experience has shown that cleaning is necessary every few years, especially in the last few sections of pipe where the velocity is the lowest. This has been easily accomplished by opening the end bulkheads and flushing with as much flow as possible by diverting effluent from the other outfalls. Hydraulic calculations for the flushing operation can also be made by the procedure outlined previously. With the bulkhead opened, a large part of

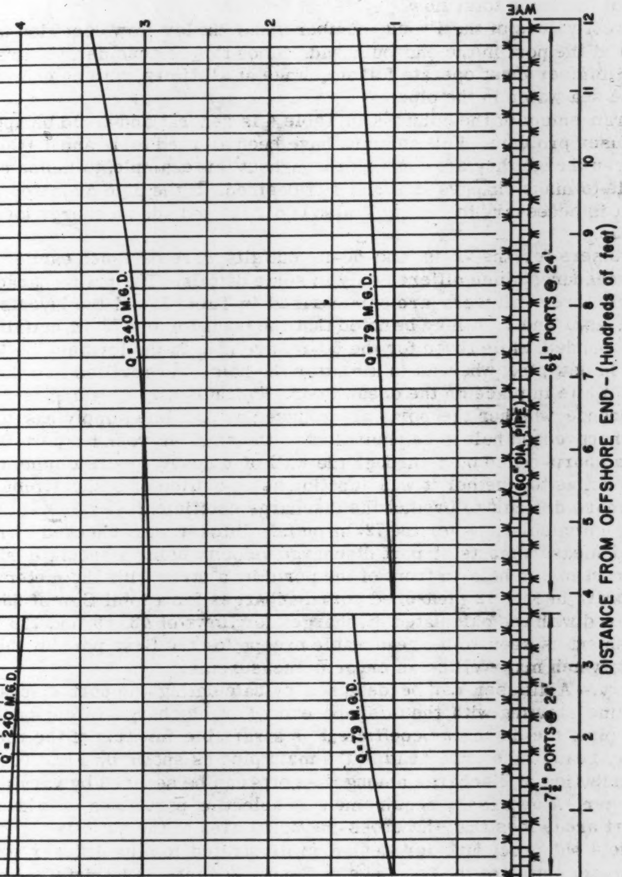

FIG. 11.—CALCULATED DISTRIBUTION OF DISCHARGE ALONG EACH LEG OF DIFFUSER FOR 90-IN. OCEAN OUTFALL

the flow discharges from the open end and pressure within the manifold is reduced; near the end, the pressure differential will be so small that the ports will discharge very little, while further back up the line the friction loss and density head will put the pipe again under positive differential pressure resulting in substantial port discharges, although much less than during ordinary operation at the same total flow.

The lowest values of the Froude number under the low flow condition is 3.2 (by Eq. 2) at the port in the end bulkhead. Since this is considerably greater than 1, the diffuser must operate full of sewage at all times, with no encroachment of the sea water in the pipe.

The arrangement of the columns in Table 4 is general and could be applied to any diffuser problem. Full columns have been allotted to D and f (Cols. 5 and 13), even though they are constant throughout this example, to indicate that it is simple to make changes in D and f if desired. If the pipe diameter D is changed, it is necessary to change V also (Col. 11) and add an energy loss for the transition.

The diffusers for the 72-in. and 60-in. outfalls were designed earlier by a similar procedure, which differed only in some details. The main characteristics of all three diffusers are summarized in Table 2 and the layouts are shown in Figs. 1 and 2. It may be noted that the ports for the 72-in. outfall diffuser are rounded, while those for the 60-in. are not. In the former, the holes were cast in new pipe, whereas in the latter the holes were drilled into the existing pipe while in place on the ocean floor. Hydraulically speaking, it makes little difference whether the ports are rounded or not; one simply has to use a larger sharp-edged hole to be equivalent to the smaller, rounded port. However, for a sharp-edged port through the wall of a heavy concrete pipe, there is some doubt as to whether it will function as an orifice or a short tube, and thus it is more difficult to predict the discharge coefficient.

The design calculations for the 72-in. outfall diffuser were checked by some crude field measurements of port discharge velocity using a specially rigged Price current meter held in front of the ports by a diver with the meter read from the boat. In Fig. 12 measured port discharges for a total flow of 53 mgd are compared with the calculated discharges for flows of 42, 89, and 138 mgd. The agreement is seen to be reasonable except for the first port on the upstream leg which may well be an error in measurement.

Summary.—A diffuser can be designed by calculating the port discharges one at a time starting with the offshore end. For discharge through lateral ports in a pipe, the discharge coefficient is a variable function of the ratio of the velocity head to the total head in the main pipe as shown by Fig. 10. Balanced distribution of discharge among the ports can be secured by varying the port diameter. A necessary requirement in selecting port sizes is to keep the sum of port areas less than the cross-sectional area of the outfall.

In Table 4 the procedure for design is illustrated for the diffuser for the 90-in. diameter outfall of the Los Angeles County Sanitation Districts. Although field confirmation of the hydraulic design is difficult, limited observations confirm the theory; the three outfall diffusers in use at Whites Point have given satisfactory service for several years.

CONCLUSIONS

1. The efficiency of disposal of primary sewage effluent to the ocean can be greatly improved by a good dispersal system.

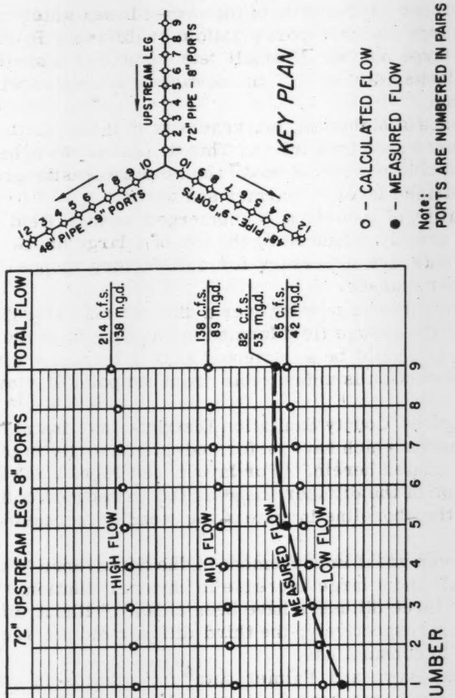

FIG. 12.—PREDICTED AND MEASURED DISCHARGES FROM 72-IN. OCEAN OUTFALL DIFFUSER

2. Increased dispersal of dilution of the sewage in sea water can most easily be achieved by a large multiple-port manifold or diffuser. By discharging the sewage flow in a large number of small jets instead of a single jet from an open-ended pipe, larger dilutions of the sewage in sea water will be achieved in the rising column.

3. In the presence of stable thermal gradients in the ocean the rising plume of sewage may never reach the surface. This will occur when the sewage mixes with sufficient cold bottom water to make the mixture density greater than that of the overlying surface layer of warmer sea water.

4. The possibility of achieving a submerged sewage field when thermal gradients exist is greatly enhanced by the use of a large multiport diffuser.

5. Ocean currents are necessary for satisfactory disposal to effect exchanges of the water masses.

6. Strong ocean currents passing over a diffuser may also give greatly increased dilution in the sewage field formed by the flow from the diffuser. The arms of the diffuser should be so oriented as to intercept a wide band of the current when its direction is unfavorable from the point of view of shoreline pollution.

7. The Los Angeles County Sanitation Districts have installed three large multiple-port diffusers which have performed satisfactorily in accordance with the principles enunciated herein. (For layout, see Figs. 1 and 2.)

8. The operation of the diffuser has resulted in reduced coliform bacteria populations along the shoreline in spite of the steadily increasing flow of sewage. (See Fig. 3)

9. There has been great improvement in the appearance in the water surrounding the outfall and a large increase in physical dilution. (See Table 2)

10. Two of the three diffusers have been cleaned without difficulty after 5 and 2 yr of service respectively; the third diffuser did not require cleaning, as it has not been in constant use.

11. The original Rawn and Palmer data[6] for dilution in a rising column from a horizontal discharge have been reanalyzed, resulting in the graphical presentations of Figs. 7 and 8. The dilution at the top of the rising column is now expressed as a function of the Froude number F and the ratio of depth to jet diameter, y_0/D.

12. A study of these relationships shows that the most feasible way to increase the dilution is to reduce the jet diameter D by use of a large number of ports, without necessarily increasing the velocity of discharge.

13. Large diffuser manifolds should be so designed that (a) the distribution of flow between ports is fairly uniform; (b) the velocities within the diffuser pipes are sufficient to avoid undue deposition; (c) the structure is easily cleanable; (d) the system operates full of sewage without sea-water intrusion; (e) the total additional head loss is reasonable; and (f) the ports themselves are of a simple design.

14. For a diffuser constructed of large pipes with small holes along the sides the hydraulic analysis involves a variable coefficient of discharge, depending on the ratio of the velocity head to total head within the diffuser (Fig. 10). Calculations of the head-discharge characteristics for the system must start at the downstream end and proceed upstream.

15. A successful diffuser usually has an aggregate port area which is less than the outfall cross-section area.

The scope of this paper has been limited to a discussion of initial dilution and hydraulic design of diffusers together with verification and illustrations taken from the history of the outfall system of the Los Angeles County Sanitation Districts. No attempt has been made to discuss in detail the complex phenomena which take place within the sewage field once it is formed. Nonetheless, considerations of factors such as coliform mortality and sedimentation, and diffusion by natural turbulence in the ocean should be considered in any comprehensive outfall design. These subjects have been treated elsewhere.[12,13,14,15,16]

APPENDIX.—NOTATION

Text:

a_n = area of nth port;

b = initial width of sewage field;

C_D = discharge coefficient for ports (see Fig. 10);

D = diameter of sewage jet at point of discharge; or, diameter of diffuser pipe;

d_n = diameter of nth port, counting n from offshore end;

E_n = $h_n + \frac{V_n^2}{2g}$ = total head at nth port (same either side by assumption);

F = $\frac{V}{\sqrt{g' D}}$ = Froude number;

f = Darcy friction factor;

g = acceleration due to gravity;

g' = $g \frac{\Delta s}{s}$ = apparent acceleration due to gravity;

h = initial thickness of sewage field;

h_n = difference in pressure head between the inside and the outside of the diffuser pipe just upstream of nth port (expressed in feet of sewage);

h_{fn} = head loss due to friction between (n + 1) and nth port;

L_o = distance along axis of rising column from point of discharge to water surface (see Fig. 6);

L_n = distance between (n + 1) and nth port;

p = fraction of sewage (by volume) in a sewage-sea water mixture;

Q = rate of sewage discharge;

q_n = discharge from the nth port;

R = $\frac{V D}{\nu}$ = Reynolds number;

S = 1/p = dilution ratio at a point on the axis of a rising plume of sewage;

S_o = dilution S at water surface $(y = y_o)$;

S_a = U b h/Q = average dilution in sewage field;

s = specific gravity of sewage;

Δs = difference in specific gravities of sea water and sewage;

U = velocity of ocean current;

V = jet velocity;

V_n = mean pipe velocity between nth port and $(n + 1)^{th}$ port;

ΔV_n = $V_n - V_{n-1}$ = increment of velocity due to discharge from nth port (or group of ports);

y = height above center of outlet;

y_o = total depth from center of outlet to water surface;

Δz_n = change in elevation between (n + 1) and nth port (measured to center of port; positive when (n+1) port is not as deep as the nth port); and

ν = kinematic viscosity of sewage.

DISCUSSION

J. M. JORDAAN JR.[20]—For the sake of academic interest the writer would like to ask whether a deflector on each diffusor outlet, so arranged as to eject the jets downward against the ocean floor, would not effect immediate and more effective mixing of sewage effluents with the colder bottom layers of the sea water. The upward rising plume is described by the authors as dipping down again at the surface, presumably because it is heavier than the surface layers, although its residual momentum first carries the effluent plume to the surface. Would it not therefore be possible to effect the mixing almost entirely by jet action so that the plume, mixed with colder, heavier water will tend to stay down?

Directing the jets vertically down will no doubt give rise to scour but they could be directed at, say, 45° down, against a protective slab covering the impact area. The jet would strike the bottom and spread out similarly to the way in which a downward directed jet of steam and vapour from a steam pipe, after striking the floor, wells up in a highly turbulent roller. A somewhat higher manifold pressure and smaller ports would be required to produce a high efflux velocity, leading to better manifold flow distribution as well. The toroidal vortex system would rise much slower around each outlet nozzle after striking the bottom and due to the central "hole" would have a greater contact zone over which turbulent mixing can occur. The effluent will also stay in contact with the bottom longer where the orbital motion of the long period surface waves sets up a turbulent zone of a few feet in depth increasing the mixing opportunities. It seems quite possible to create a submerged sewage field as also postulated by the authors. If dilutions of 10^3 times or better could be obtained by intense jetting against pads on the ocean floor, there may be no appearance of sewage effluents above the thermocline or on the sea surface at all.

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C. H. LAWRANCE, M. ASCE.[21]—This paper has been particularly interesting to the writer, who participated in the investigations and design of the 5-mile long Ocean Outfall for Effluent Disposal for the City of Los Angeles Hyperion Treatment Plant. During the course of the design of the diffuser for the Hyperion Outfall, the engineers made use of the principles and methods presented by the authors. The authors' paper condenses a great deal of information and findings resulting from the operations of the outfalls at Whites Point, yet the presentation ably presents the most important findings and conclusions.

The writer wishes to underscore the statement made by the authors' that the advantage of higher initial dilutions achieved with diffusers is partially lost because the dilution rate occurring in dilute sewage fields formed by diffusers is slower than that occurring in the narrow field originating from an open pipe. This is quite evident upon examination of the mathematical formulations of Rawn and Palmer[6] and of Brooks,[15,16] and is also borne out convincingly by field observations. It applies both to the matter of physical dilution, as may be computed from chemical and physical measurements, and to "apparent" dilution, as may be measured by bacterial densities.

A case in point is the experience of the Los Angeles County Sanitation Districts themselves upon completion of the first two diffusers for the 60-in. and the 72-in. outfalls, respectively. During the course of the rather extensive investigations undertaken by the Engineers incidental to the design of the Hyperion 5-mile outfall, the writer had occasion to analyze the laboratory data from a regular State Department of Public Health sampling, at the beach sampling stations, in the general vicinity of the Whites Point outfalls, for a control period before the construction of the diffusers, and for a similar period following completion of construction of these diffusers. For the period of January 7, 1952, through January 27, 1953, most probable numbers of coliform organisms at individual sampling stations were compiled and geometric means computed. Similarly for the period April 1, 1954, through December 20, 1954, representing the first period of use of the new diffusers, beach sampling station coliform densities were also compiled and geometric means computed.

Now the provision of diffusers on these two outfalls had resulted in a substantial improvement in initial dilution of the plant effluent upon submarine injection. The authors' Table 2 indicates that about 7.4 to 7.5 times as much initial dilution, average for the two outfalls, was obtained by the use of diffusers as was obtained from the open outfall pipes prior to these diffusers. It is noteworthy that the coliform densities at the shore stations did not decrease with corresponding magnitude. The decrease ranged from 1.40 times at the most distant station (Station 0, 24,000 ft oblique distance from the outlets) to 5.15 times at Station 3, the origin of the ocean outfalls. The average decrease for all stations studied was by a factor of 2.63. This compares with the increase in initial dilution of about 7.4.

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[21] Proj. Engr., Koebig and Koebig Cons., Engrg. Architecture, Los Angeles; Formerly San. Engr., Hyperion Engrs., Los Angeles, Calif.

The new 5-mile Hyperion Ocean Outfall for Effluent Disposal terminates in a large diffuser in nearly 200 ft depth of water and discharges an unchlorinated mixture of primary effluent and standard rate activated sludge effluent into Santa Monica Bay. The outfall and diffuser were completed in the early part of 1960, and have been in general use since that time. It is gratifying to find that their performance has so far come up to expectations regarding initial dilution, subsurface stratification of the field, and coliform densities, both offshore and at the beach stations. Piezometer connections were incorporated into the design at the halfway point in the outfall, at the beginning of the diffuser, at the point of pipe size reduction in each diffuser leg and at the terminus of each diffuser leg. Initial measurements of pressure made at these points from surface vessels and from the stationary laying barge prior to acceptance of the construction job indicated general agreement with anticipated hydraulic conditions. Additional data will be obtained from time to time in the future by the City of Los Angeles Bureau of Sanitation. These data, particularly those obtained after flows are substantially increased and piezometric pressures are higher, will be of value in establishing the hydraulic performance of the diffuser.

A M RAWN,[22] HON. M. ASCE, F. R. BOWERMAN,[23] M. ASCE and NORMAN H. BROOKS,[24] M. ASCE.—Mr. Jordaan has suggested the possibility of directing the sewage jets downward, instead of horizontally, to increase the efficiency of mixing with bottom waters. But as also pointed out by the discusser, elaborate precaution must be taken to protect the ocean bottom from scour that might undermine the pipe itself. It is doubtful whether the additional benefits would warrant the additional costs. If greater dilution is necessary, the cheapest method for achieving it is most likely to be the provision of a greater number of ports of smaller diameter.

Furthermore, the amount of dilution is limited by the amount of clean diluting water drawn into the mixing area. Intense but localized turbulence may not effectively increase dilution but only produce a more homogeneous sewage field. An essential feature of a diffuser is the wide dispersal of the effluent to limit mutual interference between rising columns (or plumes) and make vast quantities of uncontaminated dilution water readily available for entrainment.

Many different outlet devices were considered and tested during the investigation, including nozzles, long and short slots, circular manifold chambers, aspirators and jet pumps, swirl devices, baffles, and the like. Unit for unit many devices could be shown to provide more rapid mixing than a simple port, but to achieve any specified result the provision of a sufficient number of ordinary ports is cheaper, simpler, and easier to maintain over long periods in a submerged condition than a smaller number of more complicated devices.

Mr. Lawrence has clearly and correctly emphasized that when diffusers are used the rate of further natural dilution in the sewage field is decreased. This problem has not been analyzed in this paper, but a comprehensive analysis of the sewage field has been presented elsewhere.[16] However, with multi-port diffusers, there is a net increase in dilution at the shore, but proportionately less than at the outfall termini. Basically, a larger share of the ultimate mixing has been put under man's control, providing definite protection against contamination that might result from unusually rapid shoreward drift of poorly diluted sewage.

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[23] Asst. Chf. Engr., Los Angeles County Sanitation Dist., Los Angeles, Calif.

[24] Assoc. Prof. of Civ. Engrg., California Inst. of Tech., Pasadena, Calif.; Cons. to Los Angeles County Sanitation Dist., Los Angeles, Calif.

Benefits, therefore, should not be evaluated simply by averages without regard to dispersion of values. The magnitudes of the counts that are in violation of the standard of 10 coliform organisms per ml should be considered. For example, a violation of 1,000 organisms per ml is more serious than one of 50 organisms per ml. For the discharge from the diffusers operated by the Los Angeles County Sanitation Districts, the highest coliform counts are less than 1/10 of what the extreme values would otherwise have been. (It is difficult to analyze high counts quantitatively because many are indicated only by all positive tubes in the dilution tube technique when tube dilutions are set up in expectation of getting resolution for smaller coliform counts.)

Since the earlier analysis of data, terminating with calendar year 1958, additional information concerning the operation of diffusers has been tabulated for calendar years 1959 and 1960. Table 5 shows the average sewage flow thru the outfall system for the years 1957 through 1960, and shoreline bacterial data for the 4-yr period during which the two larger diffusers have both been operating. It should be noted that the average sewage has increased from 195,000,000 gal daily in 1957 to an average of 266,000,000 gal daily during 1960, an increase of 37%. During the 3-yr period 1957-1959, the annual percentage of coliform counts exceeding 10 per ml at Station 3, the closest to the outfalls, decreased from 26% to 11%. (For locations of shore stations see Fig. 3.) The data for 1960 are not directly comparable with earlier data, for commencing in 1960, the Sanitation Districts have operated a disinfection system using direct liquid chlorination during those short periods of the year when conditions in the receiving waters are such as to inadequately protect the shoreline from receiving sewage effluent-sea water mixtures in concentrations that exceed the requirements of the Regional Water Pollution Control Board. Periods requiring chlorination of the primary effluent have been determined to coincide with the following co-existing unfavorable conditions:

TABLE 5.—COLIFORMS AT SHORE STATIONS SUBSEQUENT TO INSTALLATION OF DIFFUSERS

Annual Percentage of Coliform Counts Exceeding 10 per Milliliter at Shore Stations (See Fig. 3)						Average Annual Effluent Discharge, in Million Gallons Daily
	Station					
Year	1	2	3	4	5	
1957	2	4	26	22	6	195
2958	11	9	15	5	13	218
1959	2	7	11	6	6	248
1960[a]	4	12	8	10	5	266

[a] Intermittent Chlorination Commenced During 1960.

1. Onshore winds having velocities exceeding about 10 knots and persisting for twelve or more hours; and

2. A temperature spread between the ocean water at the surface and at the bottom of the ocean in the vicinity of the diffusers of less than 6 F.; that is, essentially an isothermal ocean, without the thermocline necessary to the submergence of the sewage field.

Chlorination is utilized only in those instances in which oceanographic and meteorologic conditions limit the capability of the diffusers to disperse the sewage effluent within safe limits. During 1960, chlorination was deemed desirable for a total of 70 days, some 19% of the time; conversely, 81% of the time the outfall system satisfactorily disposed of an average of 266,000,000 gal daily of primary effluent without benefit of chlorination and without detriment to the ocean shoreline.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 3186

RAISING TRANSMISSION TOWERS WITH ENERGIZED LINES

By Albert G. Masters,[1] and Ernest J. Gesing[2]

SYNOPSIS

The demand for power makes it necessary to build additional lines and also to reconductor existing lines with larger wires. Towers on existing lines are increased in height by adding an extension to the bottom by this method to maintain code ground clearance under new and larger conductors.

INTRODUCTION

The increasing demand for power makes it necessary to build additional lines and also to increase the capacity of existing lines by reconductoring with larger wires.

When the decision has been made to reconductor an existing line, some of the towers must be increased in height to maintain code ground clearances under the new and larger conductors because of the increased sag.

There are several methods in use for raising towers. None of them are simple and all are expensive. In most cases the lines must be de-energized and the work done on week ends with special mobile-rented equipment.

The method recently developed by the writers' company and presently used for tower extension work will be discussed. This method was found to be most successful. Manhours ran about 250 per tower extension and will decrease as the crews become more experienced. All material is completely re-useable and standard line trucks and equipment are used. Lines remain energized unless it is convenient to de-energize them and the work can be

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Note.—Published essentially as printed here, in October, 1959, in the Journal of the Power Division, as Proceedings Paper 2208. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] General Superv. Engr., The Cleveland Electric Illuminating Co., Cleveland, Ohio.

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done without critical scheduling. As a safety precaution, towers are not raised when the wind exceeds 15 mph normal to the line.

DESCRIPTION OF TOWERS

The transmission towers are part of 132 kv transmission system (see Figs. 1 and 2). They are of the double circuit variety, with the bottom crossarm from 50 ft to 80 ft above grade. The crossarms are spaced 13 ft apart and the ground wire attachment is 6 ft or 11 ft above the top crossarm. These towers vary in over-all height from 90 ft to 115 ft. They are 5 1/2 ft square at the top and the bases vary from 15 1/2 ft to 20 ft square at the ground line.

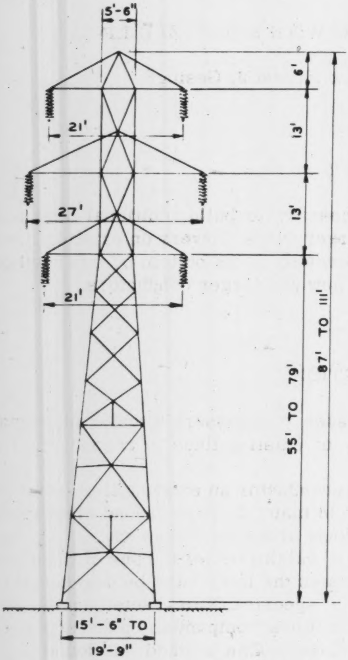


FIG. 1.—LINE TOWER

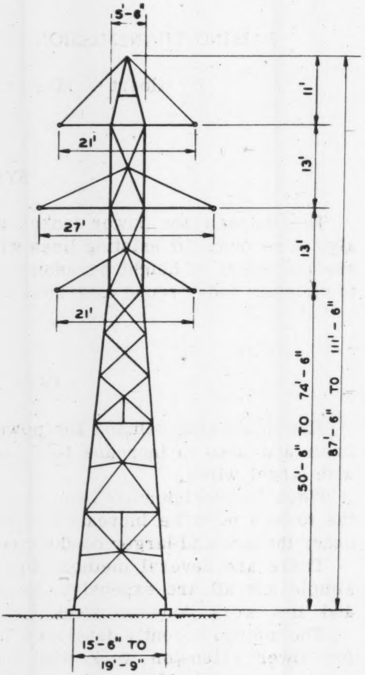


FIG. 2.—ANCHOR TOWER

The four faces of the towers are similar and are made of structural steel angles for the main tower legs, horizontal struts, and diagonal compression or tension members.

The foundation consists of stub angles attached to steel angle grillage buried directly in the ground or of stub angles encased in concrete foundations.

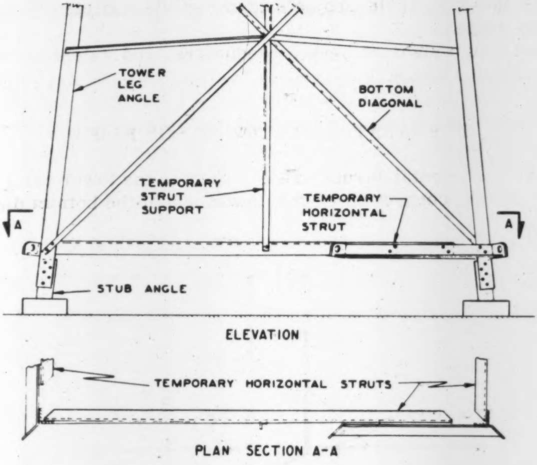

TOWER LEG ANGLE
BOTTOM DIAGONAL
TEMPORARY STRUT SUPPORT
TEMPORARY HORIZONTAL STRUT
A
A
STUB ANGLE
ELEVATION
TEMPORARY HORIZONTAL STRUTS
PLAN SECTION A-A

FIG. 3.—TEMPORARY STRUTS ON TOWER

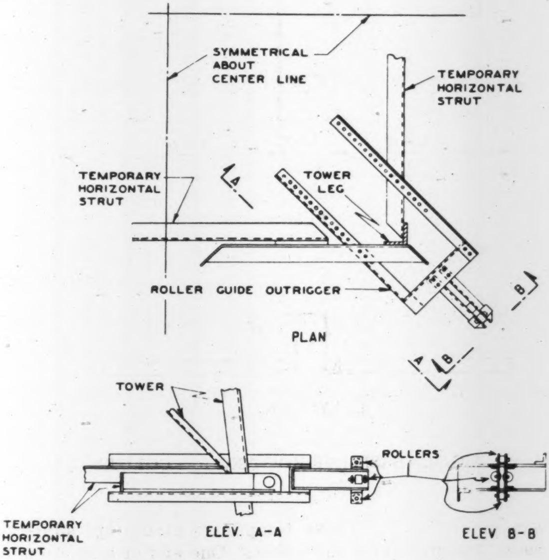

SYMMETRICAL ABOUT CENTER LINE
TEMPORARY HORIZONTAL STRUT
TEMPORARY HORIZONTAL STRUT
TOWER LEG
A
ROLLER GUIDE OUTRIGGER
B
PLAN
A
B
TOWER
ROLLERS
TEMPORARY HORIZONTAL STRUT
ELEV. A-A
ELEV B-B

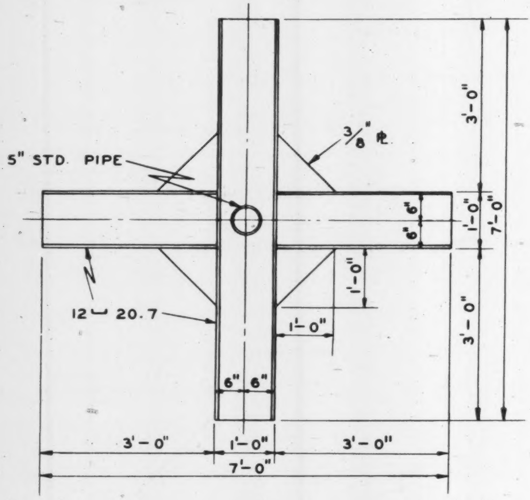
FIG. 4.—ROLLER GUIDE OUTRIGGER

In all cases the stub angle projects above grade sufficiently for splicing to the tower leg angles.

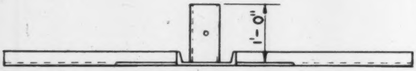
The maximum weight of tower, conductors, and extension steel is estimated to be about 12 tons.

PREPARATION OF TOWER FOR RAISING

Temporary horizontal struts are attached to the tower using the existing bolt holes at the connection of the lower end of the bottom diagonal to the tower leg angle (see Fig. 3). These temporary struts are adjustable to suit the various base dimensions of the towers. One end of each temporary strut has a 2 1/2-in. diameter hole to engage the hook of the hoisting rig snatch block. This provides a lifting point at each corner of the tower.



PLAN



ELEVATION

FIG. 5.—BASE FOR HOISTING RIG COLUMN

A roller guide outrigger, Fig. 4, is then attached to the temporary horizontal struts at each corner. These outriggers extend out and away from the tower at an angle of 45° to the face of the tower.

These roller guide outriggers were designed to act merely as a means of gently engaging the main pipe columns of the hoisting rig. Any deflection or distortion of an outrigger indicates that the raising of the tower is not proceeding in a smooth, steady, and level manner. Excessive deflection or distortion at one guide would cause an opposite guide to become disengaged and would result in an unbalanced loading on the various parts of the hoisting rig.

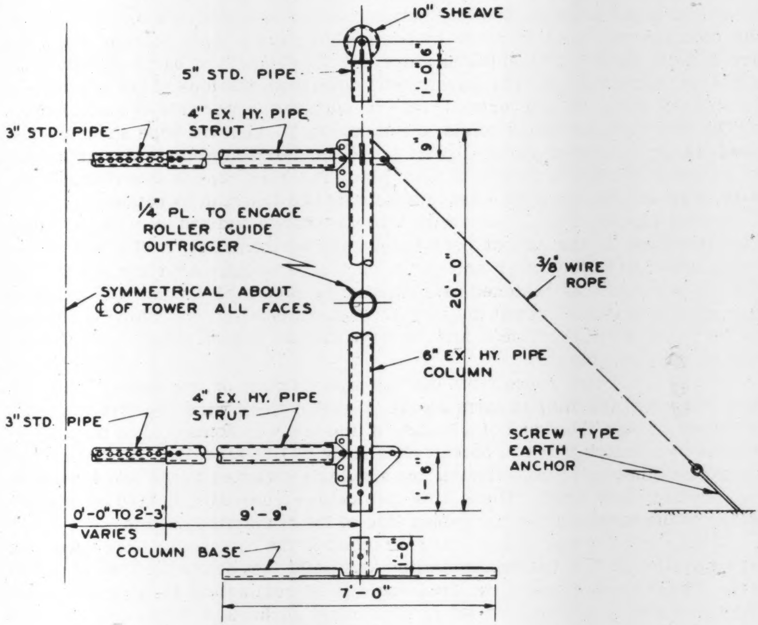

10" SHEAVE
5" STD. PIPE
9"
1'-0"
3" STD. PIPE
4" EX. HY. PIPE STRUT
6"
1/4" PL. TO ENGAGE ROLLER GUIDE OUTRIGGER
20'-0"
3/8" WIRE ROPE
SYMMETRICAL ABOUT ℄ OF TOWER ALL FACES
6" EX. HY. PIPE COLUMN
3" STD. PIPE
4" EX. HY. PIPE STRUT
SCREW TYPE EARTH ANCHOR
1'-6"
0'-0" TO 2'-3" VARIES
9'-9"
COLUMN BASE
1'-0"
7'-0"

FIG. 6.—HOISTING RIG COLUMN WITH PIPE STRUTS AND GUYS

DESCRIPTION AND ERECTION OF HOISTING RIG

The ground is leveled at each corner of the tower and the four bases for the main columns of the hoisting rig are placed in position, Fig. 5. The base consists of two 12-in. steel channels welded together to form a cross. The flanges of the channels toe up to produce a flat, smooth bearing surface. At the intersection of these two channels, a 12-in. length of 5 in. diameter pipe was welded to stand in a vertical position. The bases were designed to keep soil bearing pressures below 1,000 psf.

Twenty foot high, 6-in. pipe columns are then placed in position over the 5-in. pipes on the bases. The columns are turned to such a position that will cause a continuous 1/4-in. plate, that is welded to the pipe columns, to engage the roller end of the roller guide outriggers previously attached to the tower. The columns are then tied together, at the top and bottom, by 4-in. pipe struts. These struts are spliced at their center in such a manner as to make them adjustable to fit any spacing of columns (see Fig. 6).

There is no internal bracing between columns and struts, but the tops of the columns are held in place by two anchor guys at each column. The guys are 3/8-in. wire rope and the anchors are the screw type earth anchors with a 3/4-in. diameter rod. The guys extend down from the tops of the columns at an angle of about 45° in a vertical plane through the center line of the columns.

On the tops of the columns are mounted 10-in. wire rope sheaves. The sheaves are mounted vertically on a 1/2-in. horizontal plate that is welded to one end of a 12-in. length of 5-in. pipe. The 5-in. pipe is inserted into the 6-in. pipe column, thus the sheave assembly has freedom to rotate.

Snatch blocks, Fig. 7, each with a 10-in. sheave and swivel hook, are then installed, one at the end of each temporary horizontal strut that was previously attached to the tower and one to each pipe column near the base. Then 5/8-in. wire rope is fastened near the top of each pipe column and threaded through the snatch blocks at the ends of the horizontal struts, over the sheaves at the top of the pipe columns and down through the snatch blocks near the bottom of the pipe columns.

The 5/8-in. wire ropes from the two pipe columns on one side of the tower are connected together to form a yoke, Fig. 8. A third wire rope from a winch mounted on the front end of a standard line truck is connected to the yoke by means of a snatch block to permit equalizing of line tensions. The wire ropes from the other two pipe columns are similarly attached to the winch mounted on another line truck. These line trucks are generally located on opposite sides of the tower and on the center line of the transmission line.

Manila rope lines, Fig. 9, are attached to the two faces of the tower that are parallel to the transmission line at a point just below the bottom crossarm. These lines extend down at an angle of approximately 45° to grade where they are gently snubbed on some convenient anchorage. Their purpose is to control tilting of the tower in a direction at right angles to the transmission line. The conductors themselves control tilting in the direction of the transmission line.

OPERATION OF HOISTING RIG AND INSTALLATION OF TOWER EXTENSION

The wire rope lines to the truck mounted winches are now made taut and the weight of the tower is transferred to the hoisting rig and winch lines by disconnecting the bolts at the connections of tower legs to the anchor stub angles. The winches then raise the tower about 3 ft or 4 ft above grade, Fig. 10. While in this position new splice plates are attached to the bottom of the legs of the tower. Horizontal struts and diagonal bracing are attached to these splice plates to hold the tower square at its base. The extensions to the tower

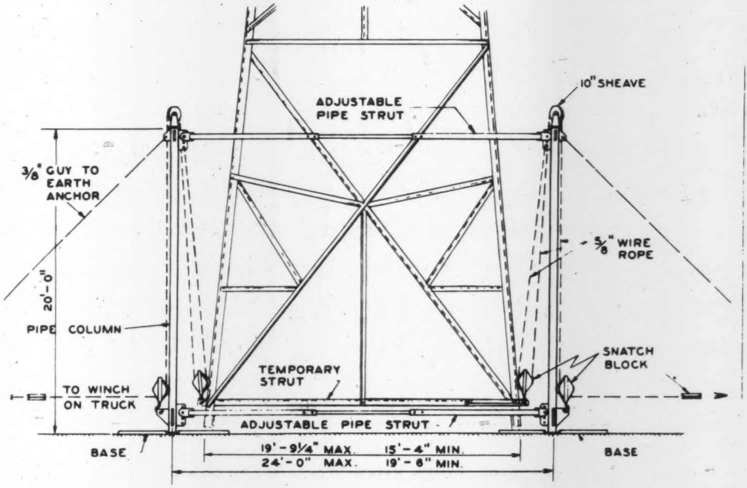

10" SHEAVE
ADJUSTABLE PIPE STRUT
3/8" GUY TO EARTH ANCHOR
20'-0"
5/8" WIRE ROPE
PIPE COLUMN
SNATCH BLOCK
TEMPORARY STRUT
TO WINCH ON TRUCK
ADJUSTABLE PIPE STRUT
BASE
BASE
19'-9¼" MAX. 15'-4" MIN.
24'-0" MAX. 19'-6" MIN.

FIG. 7.—SNATCH BLOCKS AND WIRE ROPE IN PLACE

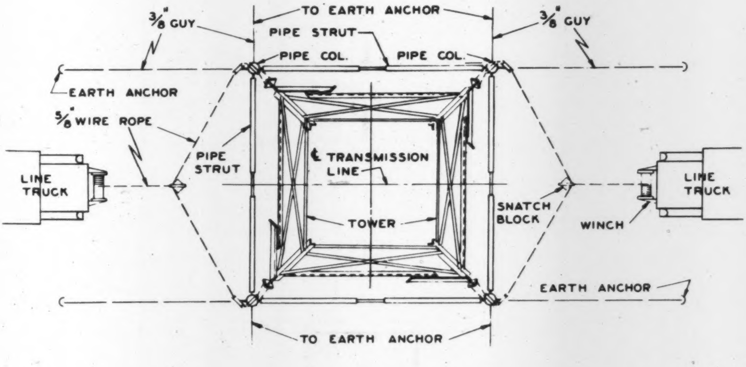

TO EARTH ANCHOR
3/8" GUY
3/8" GUY
PIPE STRUT
PIPE COL.
PIPE COL.
EARTH ANCHOR
5/8" WIRE ROPE
PIPE STRUT
℄ TRANSMISSION LINE
LINE TRUCK
LINE TRUCK
TOWER
SNATCH BLOCK
WINCH
EARTH ANCHOR
TO EARTH ANCHOR

FIG. 8.—PLAN OF WIRE ROPE LINES FROM TOWER TO WINCH

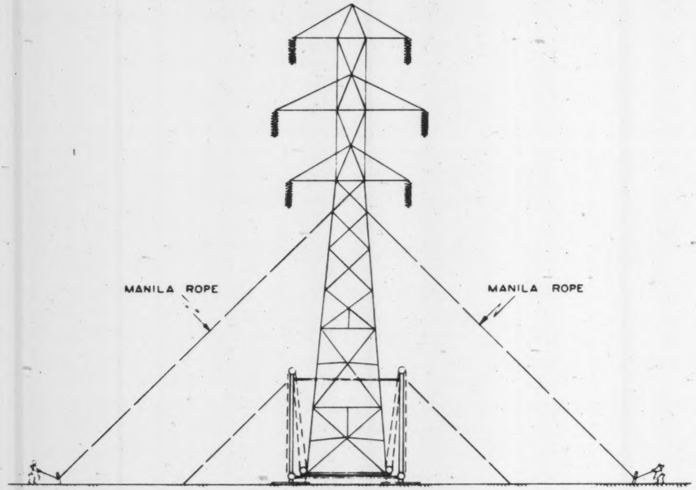

MANILA ROPE
MANILA ROPE

FIG. 9.—LINES NEAR BOTTOM CROSSARM

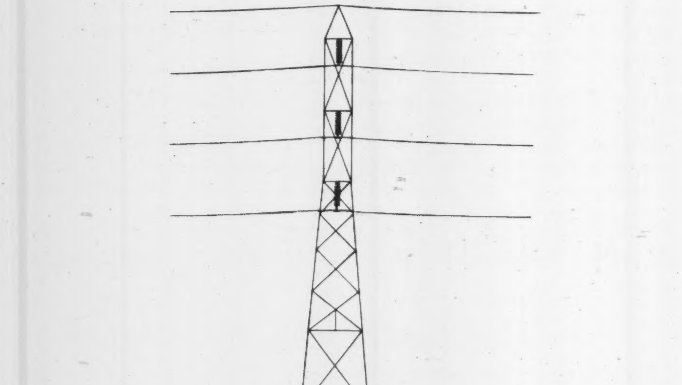


FIG. 10.—TOWER RAISED ABOUT 4 FT ABOVE GRADE

legs and other diagonal bracing are hung by one bolt to the new splice plates and horizontal struts so that they will fall into position while the tower is raised.

The truck-mounted winches then continue to raise the tower. The manila rope lines attached to the tower near the bottom crossarm are slowly slacked off as the tower is raised but continue to maintain a steadying influence on the tower. The roller guides are watched constantly to see that none of them become disengaged. As the tower rises the new leg angles of the extension swing into a vertical position and when the splice plates at the bottom of the leg angles of the extension meet the stub angles of the anchorage they are

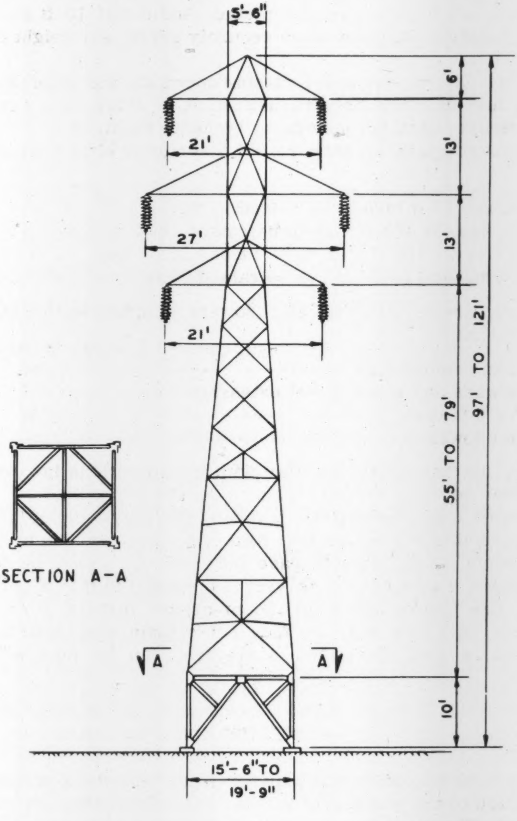


FIG. 11.—LINE TOWER WITH 10 FT EXTENSION

bolted in place. The bolting of the top of the leg angles of the extension to the tower leg angles is then completed and the remaining bracing members of the extension are installed and bolted.

The weight of the tower is now transferred to the tower extension by slacking all lines from the winches, and the hoisting rig, temporary horizontal struts, and roller guide outriggers are ready for removal.

Method of Raising a Tower and Installing a 10-ft Extension.—Fig. 11 shows a tower with a 10-ft extension completed. Additional 10-ft sections can be added to the hoisting rig, and when securely guyed, any height of lift can be made.

All material used in this tower raising operation was purchased or fabricated locally and is completely re-usable. It is stored in a company warehouse and is delivered to the job site by truck as required.

The authors' company recently raised a series of eight towers, consisting of

a. 1 line tower 87 ft high to be raised 10 ft;
b. 5 anchor towers 103 ft high to be raised 10 ft;
c. 1 anchor tower 103 ft high to be raised 20 ft; and
d. 1 anchor tower 111 ft high to be raised 20 ft.

The time required to raise all eight towers including equipment operator's time was as follows:

	Time, in manhours
Set up and remove rigs	1,180
Raise towers and install steel extension	460
Total for 8 towers	1,640
Total per tower	205

As the crews become more familiar with the operations involved, the manhours per tower continue to decrease as would be expected.

At the present time, the writers' company has four complete tower hoisting rigs. Crews are able to prepare four towers for raising, and to raise them in as short a time as possible in case the conductors must be de-energized.

The operation was found to be very successful in that it permits the use of standard line trucks and small boom-cranes instead of renting mobile-cranes. The conductors may or may not remain energized throughout the entire procedure and the work, therefore, can be done without critical scheduling.

CONCLUSIONS

The method discussed in this paper has proven to be a practical and economical solution of the problem of raising transmission towers while the lines are energized. The need for expensive and special equipment is eliminated because standard line trucks and small cranes, being available, are adequate for the work, and all component parts of the hoisting rigs are easily obtainable or can be fabricated locally.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3193

DESIGN AND SELECTION OF HYPERBOLIC COOLING TOWERS

By R. F. Rish[1] and T. F. Steel[2]

SYNOPSIS

The structural design of hyperbolic cooling towers is presented with particular reference to wind stresses in the shell. Equations are given to enable the size of cooling towers to be determined for a given cooling duty, and a method outlined for the selection of the most economic duty.

INTRODUCTION

The first hyperbolic natural draught reinforced concrete cooling tower was designed by van Iterson of the Dutch State Mines and installed at the Emma Colliery in 1916. Towers of this type were installed at Lister Drive Power Station in Liverpool in 1925 and since then have become standard practice in British power stations where cooling towers are required. Many early towers are still in use, having shown great reliability in service, little maintenance having been required apart from the replacement of decaying timber in the cooling stack. Recent advances in the fields of timber preservation and the use of alternative materials promise to extend the life of the whole of a modern hyperbolic cooling tower to the working life of its associated power station.

The largest tower so far constructed is 340 ft high and 260 ft base diameter and will cool the circulating water for a 200 MW set. The greater part of this structure is the empty shell, but the lower portion contains the cooling stack over which the warm water is distributed by a pipe and nozzle system 32 ft above the ground. The lower portion of the shell is open to allow the air access to the cooling stack, the shell being supported on legs that are inclined to resist the shearing force due to the wind. Beneath the tower a pond is constructed to catch the falling water and return it to the circulating water system. An example of a modern hyperbolic cooling tower is shown in Fig. 1.

Note.—Published essentially as printed here, in October, 1959, in the Journal of the Power Division, as Proceedings Paper 2227. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Plant Design Branch, Central Electricity Generating Bd., London, England.

[2] Plant Design Branch, Central Electricity Generating Bd., London, England.

The function of the cooling stack is to increase the surface area between the water and the cooling air, either by breaking the water up into droplets or by spreading it over a large area in the form of films. It is important that this is achieved without offering too great a resistance to the movement of air.

As the warm water falls through the stack it gives up its heat to the air. The air leaving the stack inside the shell is lighter than the ambient air and a draught is created by chimney effect. This mechanism differs from that of a mechanical draught tower in that the cooling is dependent on the dry bulb temperature as well as the wet bulb temperature, the draught for a given wet bulb temperature increasing with decrease of dry bulb temperature (that is, being better in humid conditions). Above the distribution pipes, a spray eliminator screen is constructed to catch the fine droplets of water which would otherwise be picked up by the rising air-stream and deposited over the neighborhood. As the plume of vapor is discharged at a great height, there is no nuisance due to ground fog, nor is there any risk of the performance of the tower being affected by the recirculation of the air leaving the top of the tower.

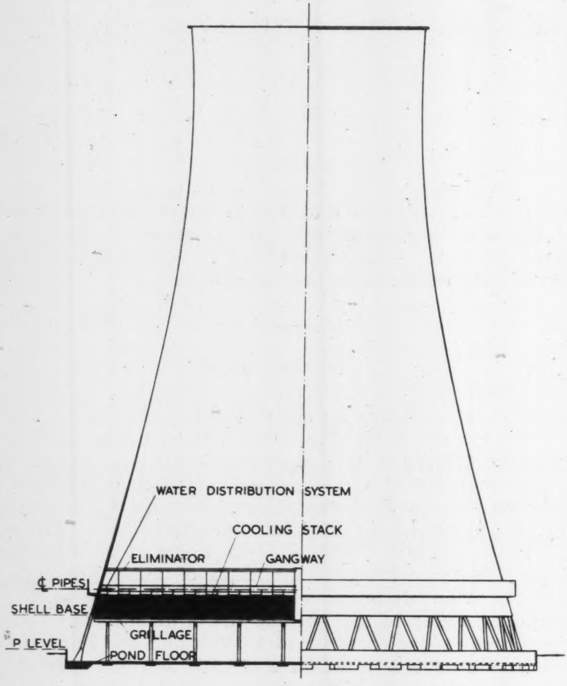


FIG. 1.—NATURAL DRAUGHT COOLING TOWER

The cooling characteristics of a hyperbolic type natural draught cooling tower are different from those of a mechanical draught installation. If it is desired to compare the economics of the two types of tower, it is necessary first to select the most economic cooling water arrangement for each type of tower by a method similar to that outlined in the paper. The most economical arrangements of both systems can then be compared taking into account such factors as overall capital cost, condenser vacuum, auxiliary power required, reliability, cost of maintenance and ground area. It can be quite misleading to compare the economics of the two types of cooling tower on the basis of a common cooling system and tower duty.

DESIGN OF TOWER SHELL BY MEMBRANE THEORY

There is no thermodynamic reason why the shell of a natural draught cooling tower should not be cylindrical. This shape simplifies the design and construction of the shell. Cooling towers have been built in Germany on these lines.

However, the momentum of the air entering the shell carries it into the center to form a vena contracta whose diameter depends on the ratio of tower diameter to height of air inlet. A considerable saving in shell surface area and volume of concrete can then be made by tapering the shell into the diameter of the vena contracta. This stiffens the shell against wind forces, and opening out the shell above the throat stiffens the shell even more.

The analysis of the effect of wind forces on a shell can be conducted by the "membrane" theory, which assumes that the thickness of the shell is so small compared with its diameter that the wind forces are resisted only by direct tensions, compressions, and shearing forces in the direction of the shell itself. The analysis of the true hyperbola of revolution is complex and it is usually conducted using a step-by-step method. A good approximation however can be made by assuming the shell to be constructed of two truncated cones with a cylinder in between. Towers of this "diabolo" shape have been built in the past, but as they are rather ugly and show no other advantage than ease of computation. They are no longer being constructed. The simple case of the cylinder demonstrates the method. The analysis of a tower constructed of two truncated cones is shown subsequently in the Appendix.

ANALYSIS OF CYLINDRICAL SHELL

Consider the small element of the shell shown in Fig. 2, affected by a normal wind pressure p, which is resisted by vertical loads per ft v and (v + dv), horizontal ring loads per ft t and (t + dt) and shearing loads per ft s and (s + ds). These forces are in equilibrium.

Resolve Vertically.

$$v\,R\,d\beta + s\,dh = (v + dv)\,R\,d\beta + (s + ds)dh \quad \ldots\ldots\ldots (1a)$$

and

$$\frac{ds}{d\beta} + R\frac{dv}{dh} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (1b)$$

Resolve Horizontally.—

$$t\,dh + s\,R\,d\beta = (t + dt)dh + (s + ds)\,R\,d\beta \quad \ldots\ldots\ldots (2a)$$

and

$$\frac{dt}{d\beta} + R\frac{ds}{dh} = 0 \qquad (2b)$$

Resolve Perpendicularly to Element.—

$$t = -\,p\,R \qquad (3)$$

Solving Eqs. 1 to 3 will give values of v, s, and t at any depth h from the top of the shell and any angle β to the wind.

From Eq. 3 $dt/d\beta = -\,R\,dp/d\beta$
Eq. 2 becomes $-\,R\,dp/d\beta + R\,ds/dh = 0$, from which $ds/dh = dp/d\beta$ and $s = h\,dp/d\beta$ + constant.

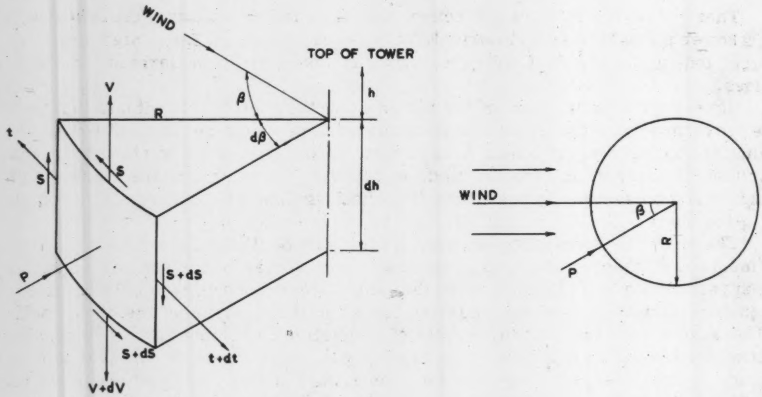


FIG. 2.—CYLINDRICAL TOWER

At the top of the shell s = 0 (that is, when h = 0, s = 0) therefore, constant = 0.

Therefore

$$s = h\frac{dp}{d\beta} \qquad (4a)$$

and

$$\frac{ds}{d\beta} = h\frac{d^2p}{d\beta^2} \qquad (4b)$$

Eq. 16 becomes

$$h\frac{d^2p}{d\beta^2} + R\frac{dv}{dh} = 0 \qquad (5a)$$

or

$$\frac{dv}{dh} = -\,\frac{h}{R}\left(\frac{d^2p}{d\beta^2}\right) \qquad (5b)$$

and

$$v = -\,\frac{h^2}{2R}\left(\frac{d^2p}{d\beta^2}\right) + \text{constant} \qquad (5c)$$

At the top of the shell v = 0 (that is, when h = 0 and v = 0) therefore, constant = 0

Therefore
$$v = -\frac{h^2}{2R}\left(\frac{d^2p}{d\beta^2}\right) \quad \text{. . . . . (6)}$$

Eqs. 3, 4(a) and (6) are then

$$t = -\,p\,R \quad \text{. (7)}$$

$$s = p'h \quad \text{. (8)}$$

in which

$$p' = \frac{dp}{d\beta}, \text{ and}$$

$$v = -\,p''\,\frac{h^2}{2R} \quad \text{. . . . . (9)}$$

in which

$$p'' = \frac{d^2p}{d\beta^2}$$

In order to solve these equations, it is necessary to know how the wind pressures are distributed around the tower shell, that is, how p varies with β.

WIND PRESSURE MEASUREMENT

Although there have been some experiments on cylinders and model cooling towers to discover the pressure distribution, in no case did the Reynolds number of the test approach that attained at high winds on a modern hyperbolic cooling tower. The difficulty was that with a model scale of 1/100 the wind velocity would have to be 7,000 mph to correspond with a 70 mph wind at the same pressure. The vibration of a chimney can extend sub-critical distributions of pressure to Reynolds numbers greater than that which would have been expected, and for this reason, it was desired to obtain readings at the highest Reynolds number attainable. A model cooling tower $26\frac{1}{2}$ in. high was constructed of sheet metal with pressure measuring points inside and outside the shell, and tested under pressure (that is, at increased density and Reynolds number) in the variable-density wind tunnel belonging to the Aerodynamics Division of the National Physical Laboratory at Teddington, England.

The distribution of wind force obtained from these tests is shown in Fig. 3. Although it would have been more accurate to use one equation to cover the whole curve, in view of the successive differentiating required, it would be easier and sufficiently accurate to split up the curve into a number of parts each covered by a simple expression. The curve consists of the algebraic sum of the internal and external wind pressures so that the value of p on the front face is made up of one velocity-head positive pressure plus approximately half a velocity-head suction inside the tower.

Example 1.—A cylindrical concrete cooling tower shell is 189 ft 4 in. diameter, 219 ft 6 in. from the top of the ring beam to the top of the tower and 9 in. thick. What are the maximum stresses produced by a uniform wind speed of 90 mph?

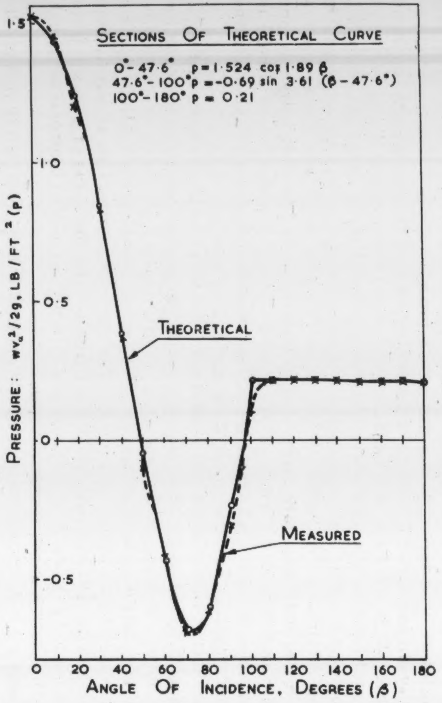


FIG. 3.—WIND PRESSURE CURVE p VERSUS β

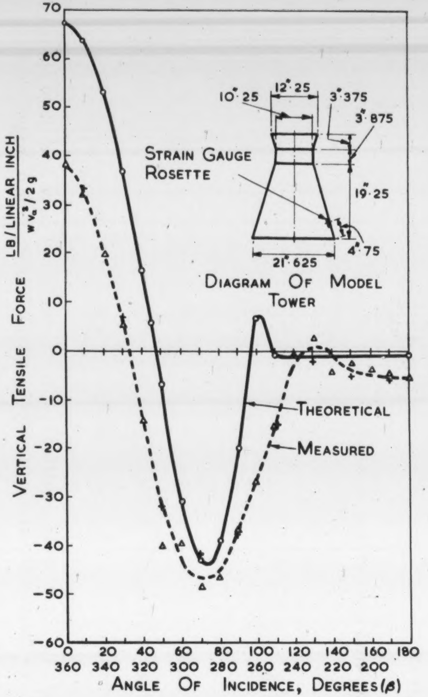


FIG. 4.—STRAIN CURVE v VERSUS β

$$t = -pR = -94.66\,p$$

$$s = p'h = +219.5p'$$

$$v = -p''\frac{h^2}{2R} = -255\,p''$$

The maximum values of t and v will occur at $\beta = 0°$ and the maximum value of s at $\beta = 47.6°$.

The wind pressure on a vertical plane surface at right angles to the wind will be $w\,v_a^2/2\,g$, in which w is the density of the air and v_a the velocity of the wind. With a velocity of 90 mph, a temperature of 59 F, and a barometric pressure of 29.53 in. of mercury, it will equal 20.75 psf.

$$p = 20.75 \times 1.524 \cos 1.89\,\beta = 31.6 \cos 1.89\,\beta$$

$$p' = -31.6 \times 1.89 \sin 1.89\,\beta = -59.7 \sin 1.89\,\beta$$

$$p'' = -59.7 \times 1.89 \cos 1.89\,\beta = -113 \cos 1.89\,\beta$$

$\beta = 0°$ $p_o = 31.6$ $p'_o = 0$ $p''_o = -113$

$\beta = 47.6°$ $p_{47.6} = 0$ $p'_{47.6} = 59.7$ $p''_{47.6} = 0$

t at $\beta = 0°$ = 2990 lb per ft (compression)

s at $\beta = 47.6°$ = 13,100 lb per ft

v at $\beta = 0°$ = 28,800 lb per ft (tension)

A 9-in. shell weighs 112.5 psf, hence v dead load = 219.5 ft x 112.5 psf = 24,700 lb per ft

The maximum resultant vertical tension is 4,100 lb per ft. If the permissible tensile stress in the steel is taken as 24,000 psi, the area of vertical steel required per foot of the shell is 0.17 sq in. or 3/8 in. bars spaced at 8 in. center to center.

It is interesting to compare the preceding results with those obtained for a tower of similar height and radius at the section considered, but constructed of two truncated cones with a cylinder in between (Fig. 4).
The equations are:

$$t = -98.9\,p$$

$$s = +121.4\,p' + 5.95\,p''' + 0.214\,p^{V}$$

and

$$v = -28.5\,p - 190.2\,p'' - 18.9\,p'^{V} - 0.63\,p^{V'}$$

which gives the loads per foot with wind pressures as in the previous example.

t at $\beta = 0°$ = - 3125 lb per ft (compression)

s at $\beta = 47.6°$ = - 6143 lb per ft

and

v at $\beta = 0°$ = 13470 lb per ft (tension)

The shear and vertical stresses are reduced by more than 50% due to the "hyperbolic" shape of the shell compared with a cylinder of the same height and base diameter.

STRAIN GAUGE MEASUREMENTS

In order to check the validity of the theory, strain gauges were fixed to the shell of the model tower and readings taken under load in the wind tunnel. Owing to the difficulties of temperature compensation in the tunnel, the accuracy of the readings was not great, nevertheless it is considered that the results support the membrane theory as being a reasonable method for the analysis of cooling tower shells. The results obtained from one set of gauges, which may be regarded as typical, are shown in Figs. 4, 5, and 6 compared with the theoretical results.

MOISTURE MOVEMENT STRESSES IN SHELL

In operation the tower shell becomes saturated with water which causes it to swell. According to L. J. Murdock[3] a typical 1:2:4 concrete with a water-cement ratio 0.6 will expand 0.028%. If the outside of the tower shell is dried by a hot sun or by wind while the inside remains saturated, assuming a linear distribution of strain, the maximum tensile and compressive stresses in the section will be $1/2 \times 0.00028$ E in which E is the Young's modulus of the concrete. If E is taken as 2.5×10^6 psi, the tensile stress produced is 350 psi which may be sufficient to crack the concrete.

There are two possible approaches to this problem. The provision of 0.3 % horizontal reinforcement on the outside of the shell with a 1-in. cover will reduce the moisture movement stresses to acceptable limits. This is fairly expensive as it involves a considerable increase in the reinforcement of the shell. Also it involves placing long heavy bars on the outside of the vertical reinforcement, whereas the working platform is usually on the inside.

The alternative is to paint the inside of the shell with bituminous paint to prevent the moisture reaching the concrete. As the scaffolding used in the construction of the shell is moving upwards fairly rapidly, to avoid heavy additional scaffolding costs, the paint has to be applied while the concrete is still green. Not many paints will stand up to these conditions and great care must be taken in the selection and application of a suitable paint. Of a large number tested by G. C. Parkinson, only two proprietory brands were found to be satisfactory and these are now specified where painting of cooling towers is required.

DESIGN OF RING BEAM AND TOWER LEGS

The ring beam is the intermediate portion between the tower legs and the main portion of the shell. The thickness of this ring beam is sometimes gradually tapered from the bottom, where it has to be thick enough to accommodate the tower legs, to $4\frac{1}{2}$ in. or 5 in. of the shell. In other designs the ring beam is made of constant thickness with an abrupt transition to the shell. The stresses set up in the ring beam may be computed from the deep beam theory. The load

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[3] "Concrete Materials and Practice," by L. J. Murdock, Arnold, London, 1955, pp. 15-16.

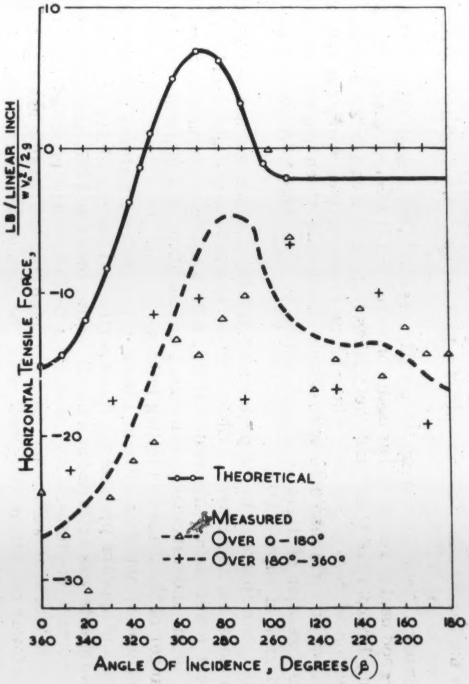


FIG. 5.—STRAIN CURVE t VERSUS β

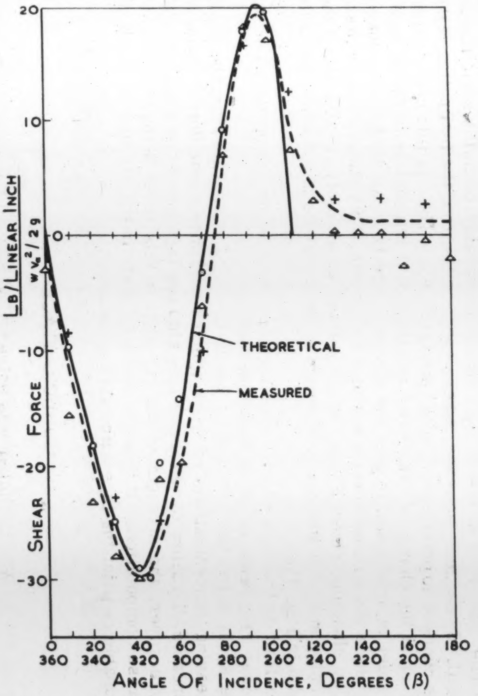


FIG. 6.—STRAIN CURVE s VERSUS β

on the inclined legs of the tower is made up of the dead load of the shell and ring beam together with the shear and vertical wind loads obtained from the membrane theory. The tower legs should preferably be in the same line as the shell. Sometimes however, where space is limited, the inclination of the legs is changed to produce a smaller pond as in Fig. 7(b). In this case tension is set up at the change of direction and although it is possible to insert sufficient steel at this point, to take the stress, there is some danger of cracking taking place due to tensile stresses in the concrete. This danger can be overcome by changing direction gradually instead of abruptly, the length of the curve being such that the tensile strength of the concrete is not exceeded. This is shown in Fig. 7(c).

The load at the base of the tower legs can be taken on the pond wall acting as a lower ring beam or on independent footings.

The spray from the circulating water tends to evaporate from the tower legs leaving concentrations of salts which sometimes cause deterioration of the concrete. To allow for attack of this kind the legs should be designed with adequate cover, the strength of the concrete in this cover being neglected in the stress computations. If there is much sulphate in the circulating water, it is advisable to paint the legs with bituminous paint.

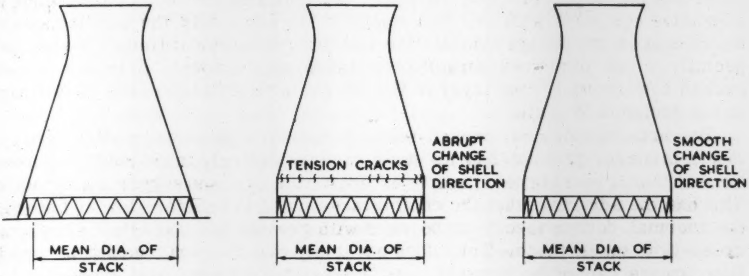
COOLING STACK

Before World War II, timber was universally used as the supporting structure for the cooling stack. Due to decay at the joints, however, the timber stacks gradually became unstable and the structures usually had to be replaced once or twice during their working life of 30 yr. Owing to the shortage of suitable timber during the war, some precast concrete supporting structures were used. It was then found that the extra capital cost was more than compensated for by the longer life. An example of a precast concrete supporting structure is shown in Fig. 8.

Timber (Pinus Silvestris) is usually used for the splash bars or lathing and a good deal of research has been conducted by the Central Electricity Generating Board (C.E.G.B.) and the timber treatment firms into the cause and prevention of decay. Suitable water borne preservatives which combine with the timber to form insoluble salts have given results which promise a considerable extension of life to cooling tower timber when used with the necessarily high degree of impregnation.

The other approach to the problem of packing decay is the use of a packing which does not deteriorate. Glass lathing has been investigated and a highly efficient tower using this material could no doubt be constructed. However, there is an understandable reluctance to use glass on a big scale because of the danger of breakage and falling lathing.

The use of asbestos-cement sheets for a packing material has also been studied and appears promising. An experimental packing of one layer of corrugated sheets was installed in a small cooling tower in 1945, again stimulated by a shortage of timber. As insufficient depth had been provided, this tower did not perform as well as had been expected, but the packing has stood up to cooling tower conditions extremely well. No sign of deterioration occurred in 13 yr of continual use. Recent analysis has shown that with a double layer of corrugated asbestos sheets, a most efficient tower should be obtained. A tower to this design was installed in association with a 120 MW set at the present


TENSION CRACKS
ABRUPT CHANGE OF SHELL DIRECTION
SMOOTH CHANGE OF SHELL DIRECTION
MEAN DIA. OF STACK
MEAN DIA. OF STACK
MEAN DIA. OF STACK

(*a*) Tower A (*b*) Tower B (*c*) Tower C

FIG. 7.—SHELL SHAPES

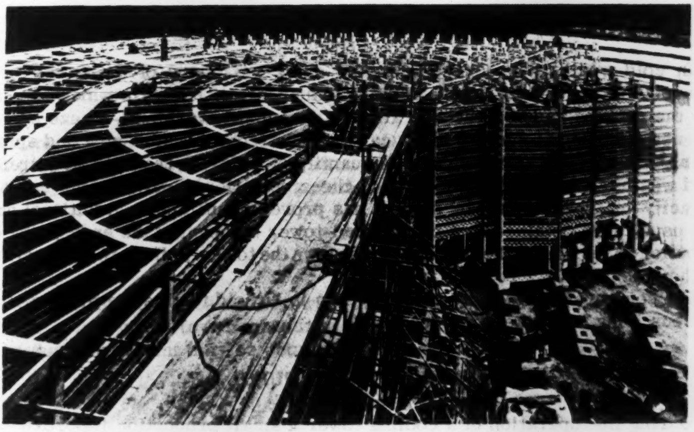
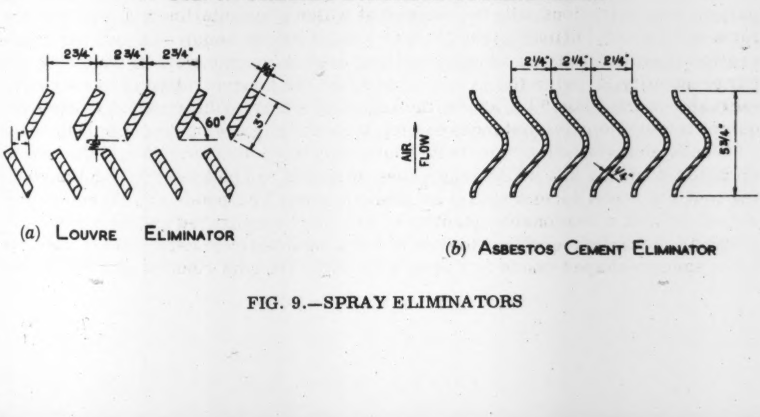


FIG. 8.—PRECAST CONCRETE COOLING STACK


2¾"
2¾"
2¾"
60°
3"
r
2¼"
2¼"
2¼"
AIR FLOW
¼"
5¾"

(*a*) Louvre Eliminator

(*b*) Asbestos Cement Eliminator

FIG. 9.—SPRAY ELIMINATORS

time; testing began in 1959. Although the asbestos-cement cooling stack is expensive compared with a timber stack for the same duty, the smaller size of tower makes the design competetive with the conventional timber design, especially if the improved durability is taken into account. Asbestos-cement packed towers using one layer of flat sheets have also been built in Germany to the design of W. Otte.

The asbestos-cement packed tower (together with some modern developments in timber packing) has its stack enclosed entirely in the shell, the tower beneath the air opening being empty except for the columns supporting the stack. This has the advantage that the cooling takes place in counter-flow and enables the thermal design theory to be used with greater accuracy than when some cross-flow takes place. This does not imply that the performance of mixed-flow towers cannot be assessed, as a great deal of empirical knowledge has been gained from past experience, but it is simpler to predict the effect of innovations in a counter-flow tower. All counter-flow towers depend for their efficiency on a low overall resistance to air flow, and it is important that an adequate air-opening should be used.

ICING

In cold weather the droplets in the outer periphery of the tower tend to freeze. Icicles develop on the packing and sheets of ice form which can damage the packing and obstruct the air flow. In most towers anti-freeze systems are now installed so that hot water can be drawn from the inlet pipe and sprayed over the periphery. A considerable quantity of water is required in extremely cold weather (approximately 25 % of the circulation), and in some cases the capacity of the anti-freeze system has proved insufficient. Additional water can usually be obtained in a center feed-tower by opening the ends of the distribution pipes where the water will run down the tower wall on to the periphery of the stack.

If a counter-flow packing is used, there is no hold for the ice to develop and serious icing is unlikely to occur.

MAKE-UP AND PURGE

During the operation of a cooling tower a small proportion (approximately 1%) of the circulating water is continually being evaporated and has to be replaced. The salts in the water are left in the solution, and if the water is not purged, concentrations will be reached at which precipitation will occur in the form of scale. Additional make-up water is therefore required above the evaporation loss, the surplus usually spilling over the purge weir to waste. If the purge quantity is twice the evaporation loss, the concentration of salts cannot increase more than 50 % above the make-up water. Chlorination is also required in certain circumstances to keep algae from growing on the splash bars.

Where an installation rejects its heat solely to cooling towers and the make-up water contains silt which may cause abrasion to condenser tubes, the cooling tower's ponds form a useful settlement area. The ponds are therefore designed so that a reasonable quantity of silt can be collected and removed.

Where a limited cooling source is supplemented by towers and the water is silty, saucer-shaped ponds are used with outlet troughs running across the diameters. Water falling from the cooling stacks will then wash any silt into the outlet troughs and keep the settlement in the ponds to a minimum.

SPRAY ELIMINATORS

The finer droplets of water produced by the nozzles and the cooling stack are picked up by the rising air-stream and can create a public nuisance if not intercepted. The problem was not tackled effectively until the nationalization of the British electricity supply industry in 1948 when a large number of eliminator screens were examined in a test tower. The most suitable was selected as the standard eliminator screen to be used in power station cooling towers. The double-layer louvred screen shown in Fig. 9(a) is low in first cost, effective in practice, and has the low pressure drop of only three velocity heads. As the column of air inside the tower is lightened by the elimination of droplets, no deterioration in performance is produced by the introduction of an eliminator screen of this type. An account of this work has been given by H. Chilton[4].

A new development has been the asbestos-cement eliminator screen (Fig. 9(b)) for which a British patent is pending. This is more effective in elimination, has a lower pressure drop and it is more durable than the timber screen, and is not more expensive in the first cost.

PERFORMANCE OF COOLING TOWERS

Chilton showed[5] that the duty coefficient D of a tower is approximately constant over its normal range of operation and it is related to the tower size by an efficiency factor known as the performance coefficient C in,

$$D = \frac{A\sqrt{H}}{C\sqrt{C}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

in which A is the base area of the tower measured at pond sill level, and H denotes the height of the tower measured above sill level.

The duty coefficient may be obtained from

$$\frac{W_L}{D} = 90.59 \frac{\Delta h}{\Delta T} \sqrt{\Delta t + 0.3124\, \Delta h} \quad \ldots\ldots\ldots \quad (11)$$

in which Δh is the change in total heat of the air passing through the tower, ΔT denotes the change of temperature of the water passing through the tower, and W_L represents the water load in lb per hr. The air leaving the packing inside the tower is assumed to be saturated at a temperature half-way between the inlet and outlet water temperatures. A divergence between theory and practice of a few degrees in this latter assumption does not significantly affect the result as the draught component depends on the ratio of the change of density to the change of total heat and not on the change of temperature alone.

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[4] "Elimination of Carryover from Packed Towers with Special Reference to Natural Draught Water-Cooling Towers," by H. Chilton, Transactions, Institution of Chemical Engineers, Vol. 30, 1952.

[5] "Performance of Natural Draught Water-Cooling Towers," by H. Chilton, Proceedings, Institution of Electrical Engineers, Vol. 99, 1952.

Example 2.—

Temperature of water to tower = 82°F
Recooled water temperature = 70°F
Temperature range ΔT = 12°F
Dry bulb air temperature t_2 = 57°F
Aspirated wet bulb air temperature t_2' = 51.7°F
Water loading to tower W_L = 38.2 million lb per hr

$$\begin{array}{llll}
t_1 = (82^\circ + 70^\circ)/2 = & 76^\circ & t_1' = 76^\circ & h_1 = 32.1 \\
t_2 = & \underline{57^\circ} & t_2' = 51.7^\circ & h_2 = \underline{13.6} \\
\Delta t = & 19^\circ & & \Delta h = 18.5 \\
0.3124\,\Delta h = & \underline{5.8} & & \\
 & 24.8 & &
\end{array}
\quad \left(\begin{array}{c}\text{from hygrometric} \\ \text{tables}\end{array}\right)$$

$$\frac{W_L}{D} = 90.59 \frac{\Delta h}{\Delta T} \sqrt{\Delta t + 0.3124\,\Delta h}$$

$$= 90.59 \times \frac{18.5}{12} \sqrt{24.8} = 696$$

$$D = \frac{38{,}200{,}000}{696} = 55{,}000$$

The performance coefficients usually attained in the past have been in the region of 5.2 in which water loadings are over 750 lb per hr per sq ft though new types of packing are bringing this down, that is, improving it.

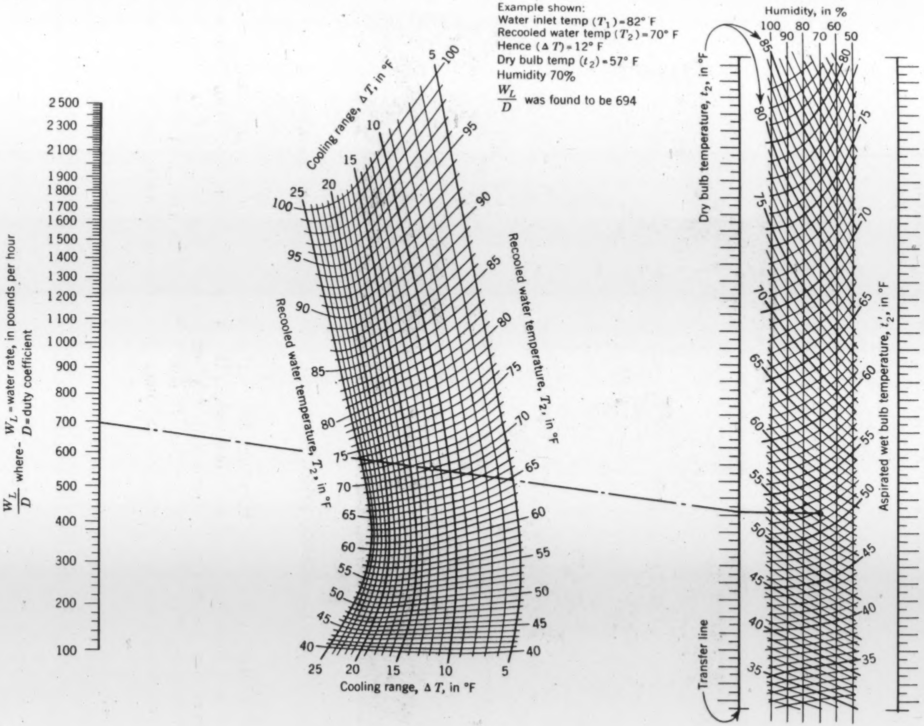
Taking a C value of 5.0 and a tower height of 320 ft the base area of the tower will be $(55{,}000 \times 5\sqrt{5})/\sqrt{320} = 34{,}600$ sq ft. This means that the internal base diameter at sill level will be 210 ft. A ratio of height to base diameter of 3:2 is normally used.

In order to find how a tower of any given duty coefficient will perform under varying conditions of air temperatures, water loadings, and temperature ranges, Eq. 11 has been plotted as a nomogram in Fig. 10.

A more detailed version of this nomogram or performance chart may be modified for a particular value of duty coefficient and included in the specification for a cooling tower. After the tower is constructed, it can then be tested within the specified range of conditions, and its success will be judged from the relation between the recooled temperature attained and that shown on the nomogram for the air temperatures, water loading, and temperature range imposed on the tower.

The C.E.G.B.'s present practice requires the guarantee to cover water loadings between 90 % and 110 % of the normal water rate, cooling ranges between 2°F below and 2°F above the normal cooling range, atmospheric wet bulb temperatures between 40°F and 60°F, and humidities between 50 % and 100 %.

It is standard practice to measure the air temperatures at 4 ft above the ground with an aspirated psychrometer, and on this basis, towers appear to vary considerably in their performance from one test to the next. As the air entering the tower actually comes from a higher level than this and the draught is affected by the air density up to the top of the tower, it is evident that more consistent results may be obtained by taking air temperatures at a higher level. The results of two tests taken on one tower are shown in Table 1. More work



Example shown:
Water inlet temp (T_1) = 82° F
Recooled water temp (T_2) = 70° F
Hence (Δ T) = 12° F
Dry bulb temp (t_2) = 57° F
Humidity 70%
$\frac{W_L}{D}$ was found to be 694

FIG. 10.—UNIVERSAL PERFORMANCE CHART FOR NATURAL DRAUGHT COOLING TOWERS

is continuing on these lines, but any rapid change in test procedure is undesirable as most of the test data available have been based on the old method, and the ambient temperature records have also been taken at a standard height of 4 ft above the ground. Owing to the variation in measured tower duty, it is advisable for the tower performance to be assessed on the average duty attained on a number of tests spread over the range of the guarantee.

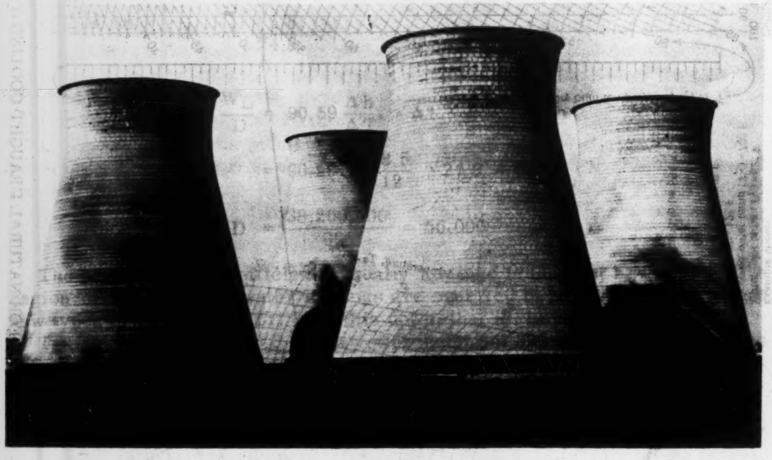


FIG. 11.—PLUME FROM NATURAL DRAUGHT COOLING TOWERS

Wind does not appear to affect the performance of towers significantly, but tests are specified to take place when the wind velocity is less than 15 mph. Towers are tested for not less than 4 hr with steady operating conditions, and the test period is taken as the hour in which conditions are most steady. If the air temperatures are rising or falling, the hold-up of water in the pond can produce serious errors in the result. Readings are taken every 5 min and all instruments have to be calibrated not more than 6 months before the test. Water flows are usually taken by pitot tube traverses of the conduits, but venturis, weirs, and heat balances across the condenser have also been used.

SPACING OF TOWERS

Figs. 11 and 12 show cooling tower installations in modern power stations, the spacing of the towers, and their relation to the rest of the plant. Towers are normally spaced with one and a half internal base diameters between their

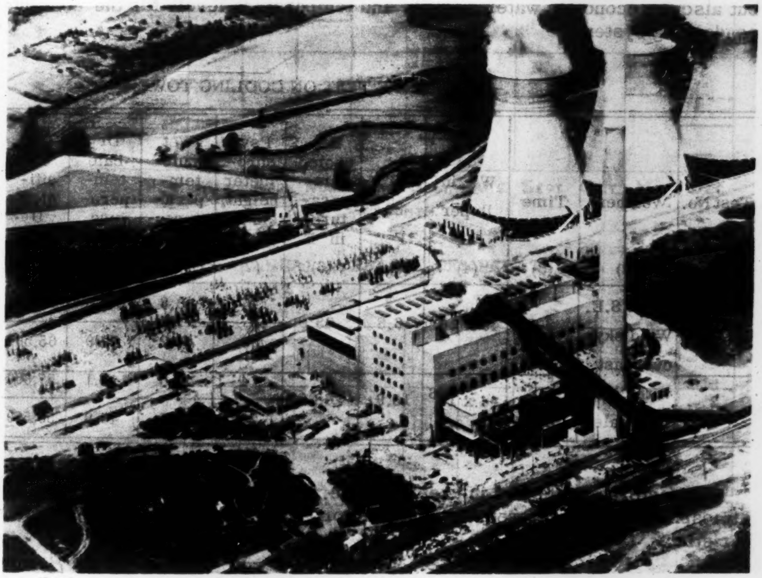


FIG. 12.—POWER STATION SITE

centers at which spacing they do not appear to influence each other significantly, however, further work into this problem is being planned.

SELECTION OF COOLING WATER DUTY

The selection of the cooling tower duty cannot be treated in isolation from the entire cooling system. Any change in cooling water quantity will be reflected in the temperature range through which the cooling towers have to recool the water, and these variations will have repercussions on the cost of the other circulation water plant items, namely, condensing plant, cooling water pumps, pipework and valves, and pumping the cooling water. This means that the investigation should not only establish the appropriate cooling tower duty, but also the economic water quantity and temperature levels for the complete condensing system.

TABLE 1.—RECORD OF TEST ON COOLING TOWER

Test No. (1)	Weather (2)	Time (3)	W_L, in lb per hr (4)	ΔT, in °F (5)	Recooled temperature, in °F (6)	Psychrometer height, in ft (7)	Wet bulb temperature, in °F (8)	Dry bulb temperature, in °F (9)	Duty coefficient D (10)
1	Light S.E. Wind sky overcast with clear intervals	2-3 (afternoon)	45×10^6	16.8	68.8	4	48.8	53.0	70,900
						20	47.2	50.9	65,500
2		1-3 (night)	44×10^6	15.9	68.5	4	45.3	47.1	57,300
						20	44.9	46.9	56,500

The factors considered in assessing the most economic tower duty are (a) atmospheric temperatures, (b) load factor, and (c) turbine exhaust characteristic.

AMBIENT ATMOSPHERIC TEMPERATURES

In Britain, the atmospheric temperatures do not differ appreciably over the general areas where adequate fuel supplies (and loading) are relatively close at hand, but where cooling facilities may well be limited. The atmospheric temperatures can, therefore, be averaged without seriously prejudicing the result of such investigation. To take an example, for medium load, that is, 2-shift operation cooling tower installations, the dry and wet bulb temperatures based on a daily period from 6 A.M. to 9 P.M. are significant. Fig. 13 shows the monthly average of the returns from thirteen meteorological stations during a period of 50 yr. When considering an installation remote from these gen-

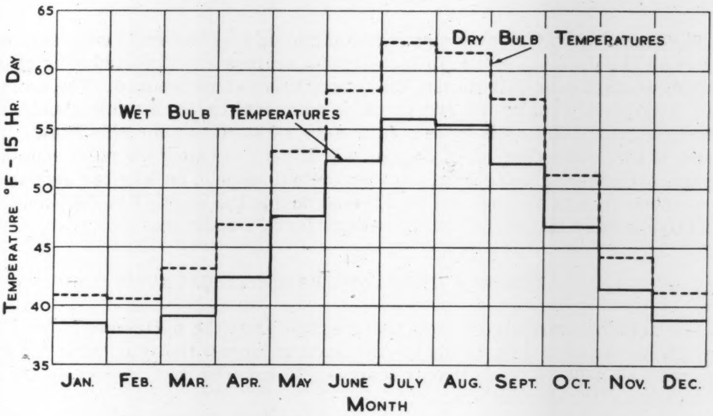


FIG. 13.—ANNUAL AVERAGED TEMPERATURES

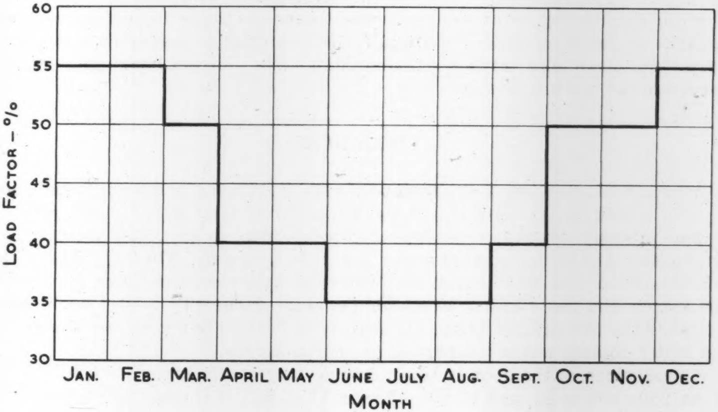


FIG. 14.—ANNUAL AVERAGED LOAD FACTORS

eral areas of development, reference can be made to the local meteorological records.

LOAD FACTOR

The plant load factor may also vary and should be given some consideration. In Britain the problem is simplified in that machines are expected to be block-loaded, which means that partial load conditions can be ignored. The average load factor can usually be predicted on an annual basis, but this should, if a more accurate solution is to be obtained, be reduced to a monthly basis. The shape of the load-curve is, of course, significant, because the most economic design, even for a given annual load factor, will depend on whether generation is concentrated in one period or diffused during the year. Fig. 14 shows the monthly average load factor for a medium load installation.

TURBINE EXHAUST CHARACTERISTIC

The turbine exhaust characteristic established the performance level for any given vacuum. Fig. 15 shows the vacuum correction curves at full load for two 120 MW turbine exhaust designs. The dotted line represents the one with the smaller exhaust area.

The average rate of loss in performance for vacua below 28.9 in. Hg is 2.4 % per in. Hg for the small exhaust, but there is little improvement in performance with higher vacua. The large exhaust on the other hand, has an average rate of loss in performance for vacua below 28.9 in. Hg of 3.4 % per in. but there is considerable improvement in performance with higher vacua. It can be quite misleading to assume a constant rate of change in performance with variation in vacuum in order to simplify the economic assessment, and furthermore, the particular turbine exhaust design has a direct bearing on the economics of the cooling system.

PROCEDURE

A better illustration of the method of establishing the economic cooling tower size would be an example of the procedure based on a 120 MW machine.

The monthly-averaged atmospheric temperatures and load factors were established and three cooling water quantities, namely, 470,000, 587,000, and 783,000 lb per min were assumed. The total heat rejection from the machine was known and the cooling water temperature range (T_2 - T_1) could be established for the various water quantities, T_1 and T_2 being the condenser inlet and outlet cooling water temperatures, respectively.

Three sizes of the cooling tower having duty coefficients of 72,400 (Tower 1), 58,250 (Tower 2), and 45,900 (Tower 3) were considered in relation to the monthly-averaged atmospheric temperatures, cooling water quantities, and temperature ranges, in order to establish, by use of the performance chart (Fig. 10), the averaged monthly re-cooled water temperatures. These temperatures are shown in Table 2 for Tower No. 2.

Three condensing plants were taken into account and were designed to cope with each of the three cooling water quantities and have the necessary heat transfer surface to give final terminal differences ($T_3 - T_2$) of 6°, 10°, and 14°F, T_3 being the vacuum steam temperature. The monthly averaged recooled temperatures, and the condenser overall terminal differences ($T_3 - T_1$) were established for each permutation of cooling tower size, water quantity, and condenser design, and the averaged monthly vacuum for each arrangement established.

TABLE 2.—COOLING TOWER RECOOLED TEMPERATURES[a]

Month	Dry bulb temperature, in °F	Wet bulb temperature, in °F	Tower No. 2
			Re-cooled temperature, in °F
$T_3 - T_1 = 22.0$ °F	Water quantity 587,000 lb per min		$T_2 - T_1 = 16.0$ °F
Jan.	40.1	38.0	60.4
Feb.	39.8	37.1	60.1
March	43.0	38.8	61.1
April	46.6	41.9	63.0
May	53.0	47.4	66.7
June	59.2	52.9	70.5
July	63.0	56.2	72.9
August	61.7	55.5	72.2
Sept.	57.7	52.1	69.9
Oct.	51.0	46.8	65.9
Nov.	43.3	40.9	62.2
Dec.	40.1	38.2	60.5
$T_3 - T_1 = 26.0$ °F	Water Quantity 470,000 lb per min		$T_2 - T_1 = 20.0$ °F
Jan.	40.1	38.0	58.2
Feb.	39.8	37.1	58.0
March	43.0	38.8	58.9
April	46.6	41.9	60.9
May	53.0	47.4	64.6
June	59.2	52.9	68.4
July	63.0	56.2	70.8
August	61.7	55.5	70.0
Sept.	57.7	52.1	67.7
Oct.	51.0	46.8	63.9
Nov.	43.3	40.9	60.0
Dec.	40.1	38.2	58.4

[a] $T_3 - T_2 = 6.0$ °F

When the averaged monthly vacuum was established the differential variation in turbine performance was obtained from a vacuum correction curve similar to those shown in Fig. 15.

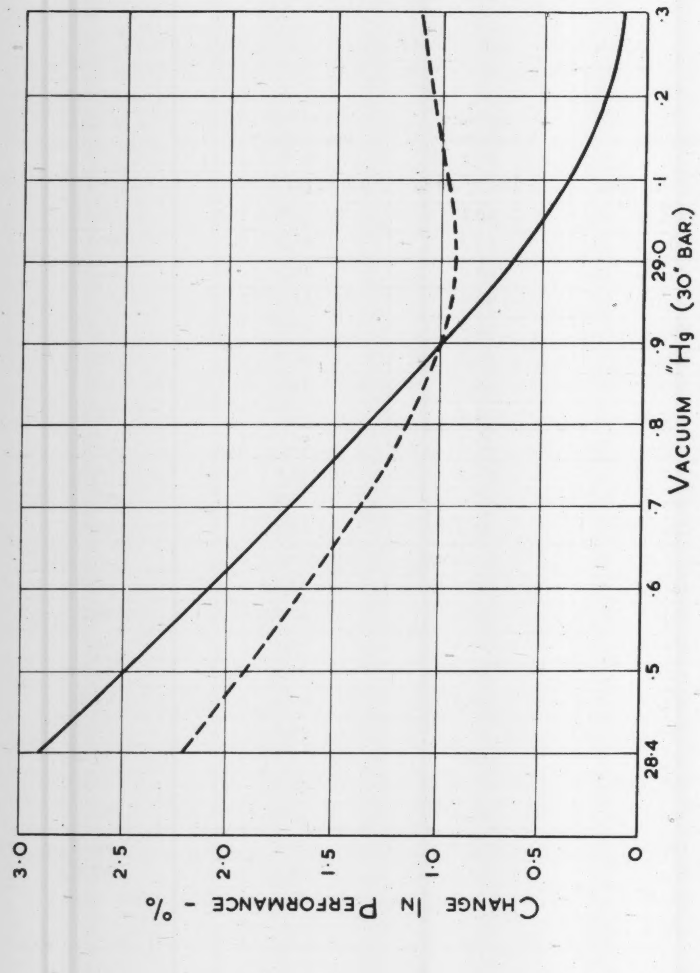


FIG. 15.—TURBINE CORRECTION CURVES

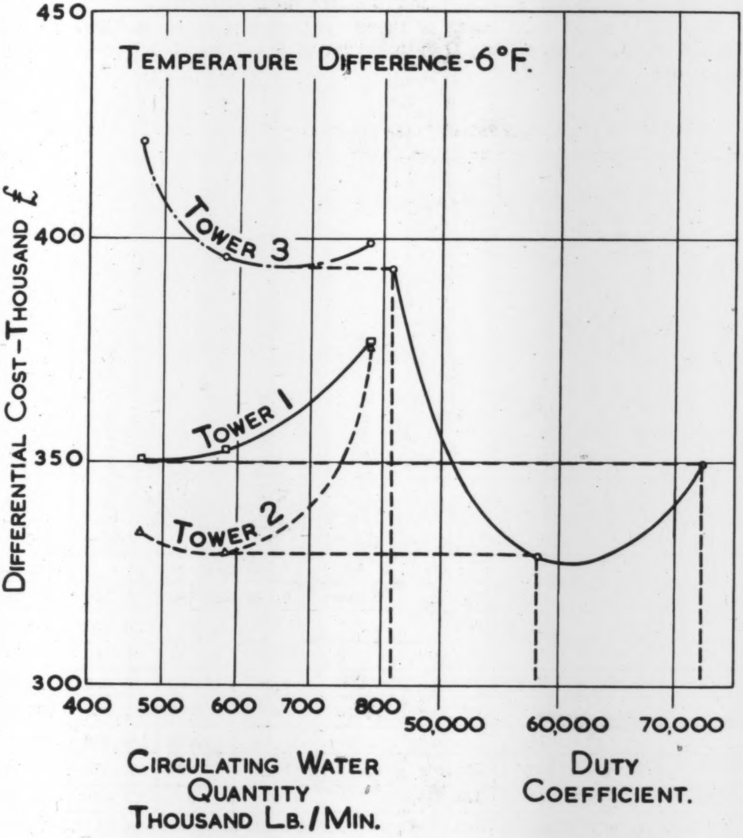


FIG. 16.—DIFFERENTIAL COSTS

The hydraulic losses of the cooling water systems were computed and the appropriate differential pumping costs were included in the assessment, together with the differential values of turbine performance and the differential capital cost of the various arrangements.

The cooling systems incorporating condensers designed for a terminal difference (T_3 - T_2) of 6° were found to be the most attractive.

The differential total costs of these systems are shown in Table 3 and in Fig. 16. A cooling tower duty D in the order of 60,000 was found to be the economic duty.

TABLE 3.—TOTAL DIFFERENTIAL COSTS OF COOLING WATER SYSTEM[a,b]

Tower size (1)	Condenser terminal difference T_3-T_2°F (2)	Water quantity, in 1000 lb per min (3)	Tower cost + fixed pumping cost (4)	Variable pumping cost (5)	Piping, culverts, pumps and valves cost (6)	Condensing plant cost (7)	Turbine running cost (8)	Differential cost (9)
1	6	783	328,000	11,460	21,145	15,700	0	376,305
1	6	587	287,700	9,335	8,880	14,200	32,150	352,265
1	6	470	265,585	3,740	0	0	80,750	350,075
2	6	783	254,000	8,350	21,145	15,700	76,050	375,245
2	6	587	203,700	7,000	8,880	14,200	95,850	329,630
2	6	470	181,585	1,870	0	0	150,150	333,605
3	6	783	216,000	5,240	21,145	15,700	140,350	398,435
3	6	587	175,700	4,665	8,880	14,200	191,850	395,295
3	6	470	153,585	0	0	0	267,350	420,935

[a] Cost given in British Pounds. [b] One Dollar is equal to approximately 2.8 Pounds.

This brief outline of the procedure for establishing the economic cooling tower size is based on an installation having no other cooling source than cooling towers. In some locations it may be attractive to use a small river and supplement its cooling capacity with cooling towers. Under these circumstances the same approach is made to the problem, but the computations become more complex because the natural temperature and the water flow variations of the direct cooling source have to be considered.

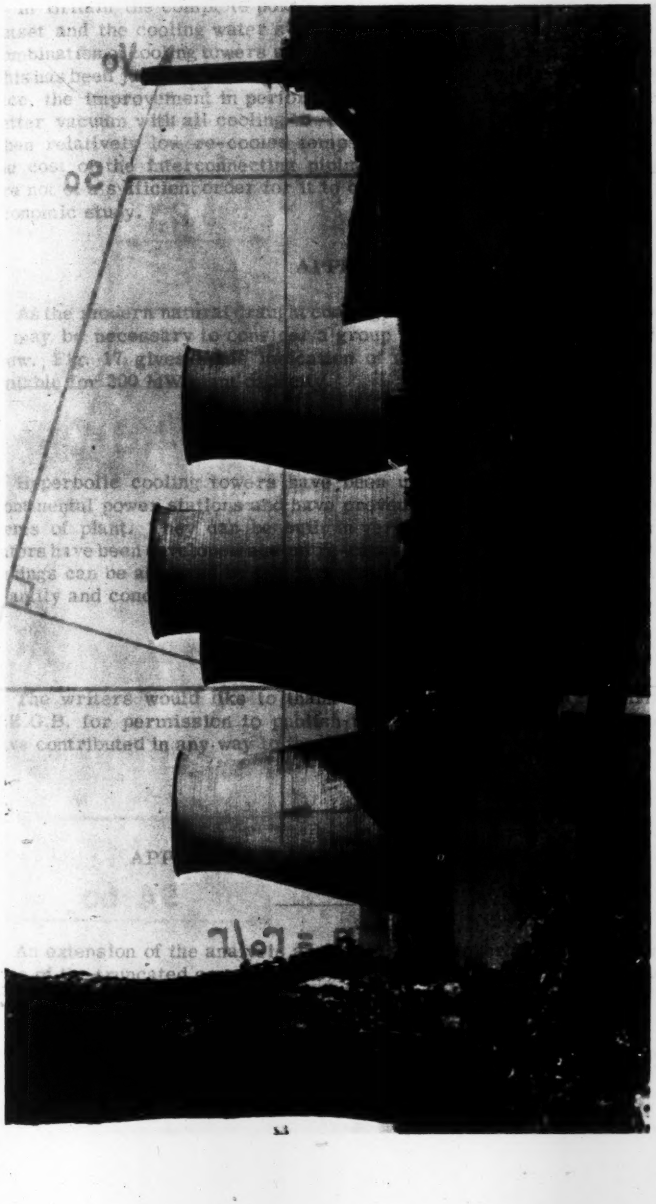


FIG. 17.—300 MW COOLING TOWER INSTALLATION

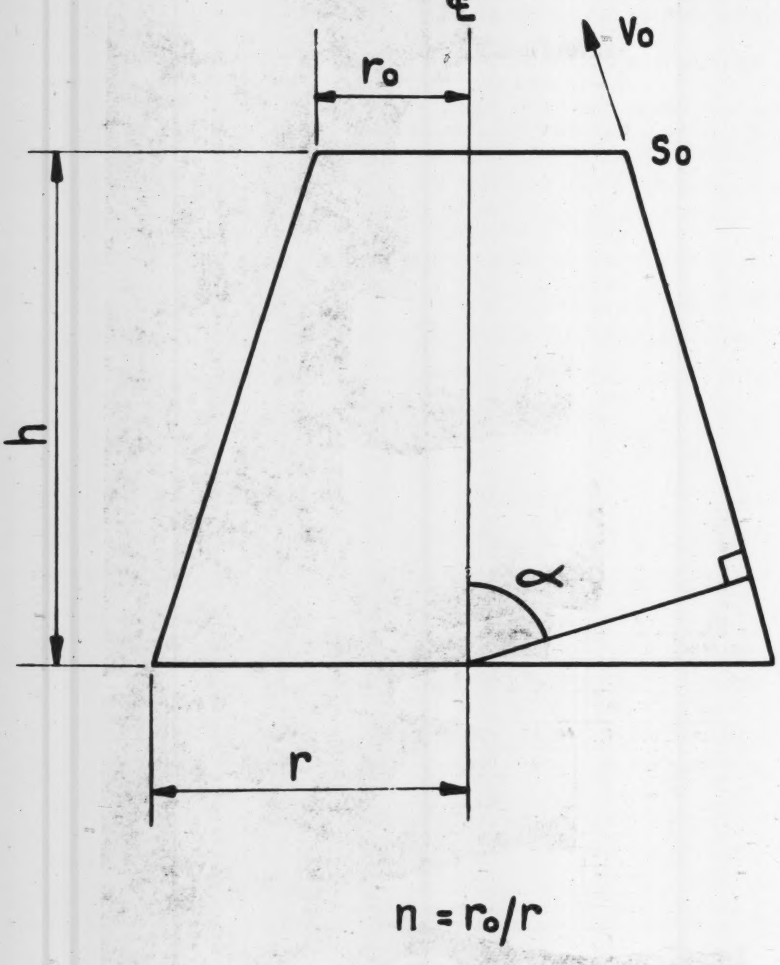

℄
r₀
V₀
S₀
h
α
r
n = r₀/r

FIG. 18

In Britain the complete power station has generally been planned from the outset and the cooling water system has been fully interconnected so that any combination of cooling towers and pumps can operate with any condensing plant. This has been justified on the grounds that when a machine has been out of service, the improvement in performance with the remaining machines, due to a better vacuum with all cooling towers in operation or reduced pumping power when relatively low re-cooled temperatures have been available, outweighs the cost of the interconnecting piping and valves. The effects of this feature are not of a sufficient order for it to be necessary to consider them for initial economic study.

APPEARANCE

As the modern natural draught cooling tower is a relatively large structure, it may be necessary to consider a group of towers from an aesthetic point of view. Fig. 17 gives some indication of appearance of a cooling installation suitable for 300 MW plant capacity.

CONCLUSIONS

Hyperbolic cooling towers have been used for many years in British and Continental power stations and have proved to be economical and trouble-free items of plant. They can be built in very large units. Effective drift eliminators have been developed and no re-circulation is experienced. Considerable savings can be achieved by the correct choice of tower duty, circulating water quantity and condenser size.

ACKNOWLEDGMENTS

The writers would like to thank F. H. S. Brown, Deputy Chairman of the C.E.G.B. for permission to publish this paper and also their colleagues who have contributed in any way to the preparation of this study.

APPENDIX.—ANALYSIS OF "DIABOLO" SHELL

An extension of the analysis applied to the cylinder to the more complicated case of the truncated cone (Fig. 18) with a direct force V_0 and shear force S_0 acting on the top leads to following relationships:

$$V = -\frac{p'' h^2 (n + 2n^2)}{6r_0 \sin^3\alpha} - \frac{p \cos\alpha h(1+n)}{2\sin^2\alpha} - \frac{s_0'(n - n^2)}{\cos\alpha} + V_0 n \quad \ldots (12)$$

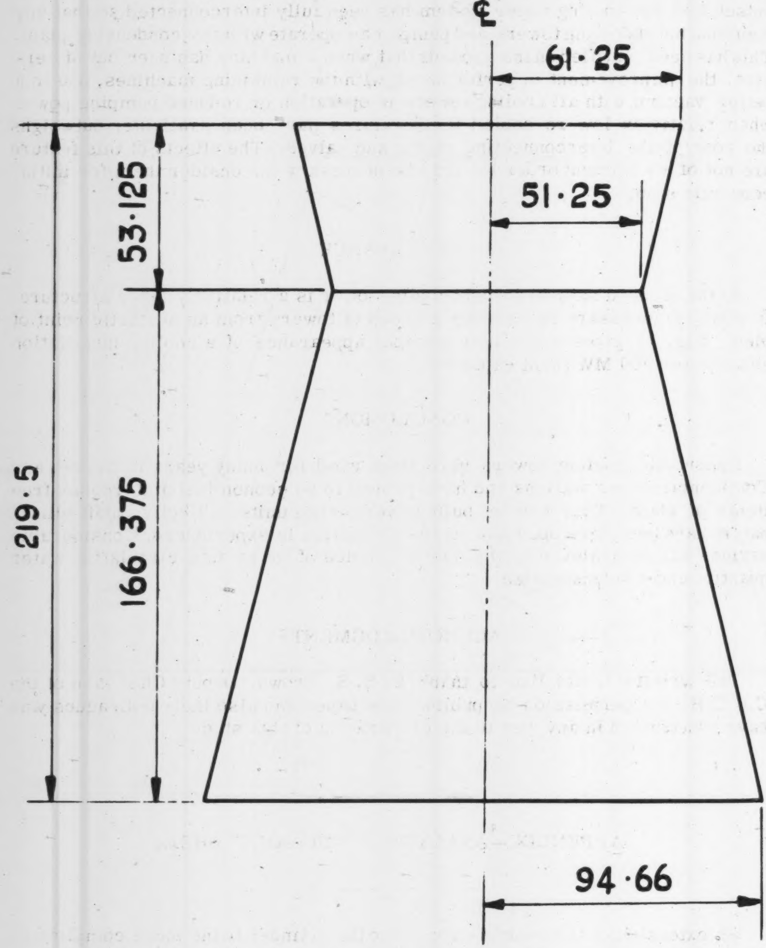

℄
61·25
51·25
53·125
219·5
166·375
94·66

FIG. 19

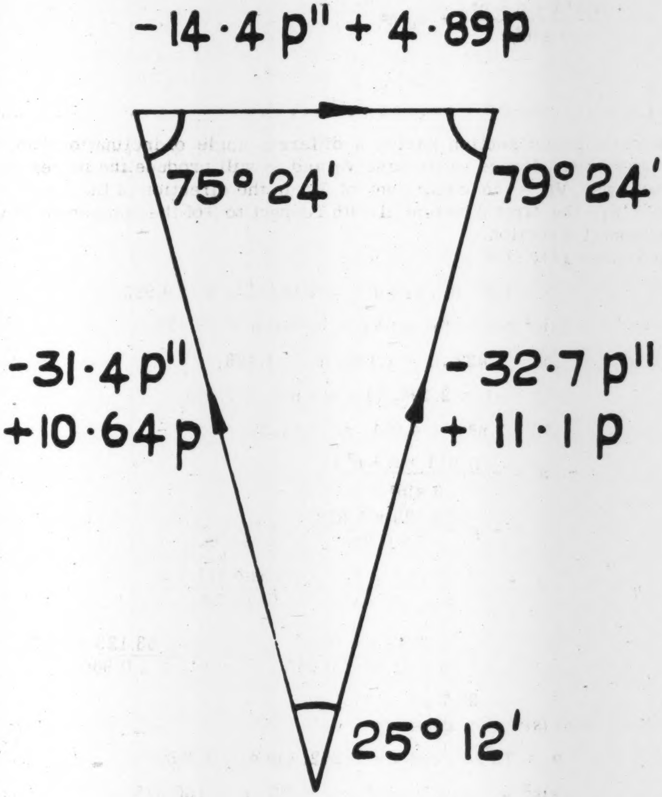

-14·4 p'' + 4·89 p
75° 24'
79° 24'
-31·4 p''
+10·64 p
-32·7 p''
+ 11·1 p
25° 12'

FIG. 20

$$S = \frac{p'h(1 + n + n^2)}{3 \sin^2\alpha} + S_0\, n^2 \quad \ldots\ldots\ldots\ldots (13)$$

and

$$t = - p\, r \operatorname{cosec} \alpha \quad \ldots\ldots\ldots\ldots (14)$$

A superimposed section having a different angle of inclination α and with direct and shear forces at its base V_1 and S_1 will produce the forces V_0 and S_0 as follows: V_0 = the component of V_1 in the direction of the lower shell, and $S_0 = S_1$, the first differential with respect to β of the component of V_1 in the horizontal direction.

Top Section (Fig. 19).—

$$\alpha = 100° \, 36', \cos \alpha = - 0.184, \sin \alpha = 0.983,$$

$$\sin^2 \alpha = 0.966, \sin^3 \alpha = 0.950, h = 53.125,$$

$$h^2 = 2822, n = 1.195, n^2 = 1.428,$$

$$(1 + n) = 2.195, (1 + n + n^2) = 3.623,$$

$$(n + 2 n^2) = 4.051, r_0 = 61.25.$$

$$S = + \frac{p'h(1 + n + n^2)}{3 \sin^2 \alpha}$$

$$= + p' \frac{53.125 \times 3.623}{3 \times 0.966} = \underline{\underline{+ 66.4\, p'}}$$

$$V = - \frac{p'' h^2 (n + 2 n^2)}{6\, r_0 \sin^3 \alpha} - \frac{p \cos \alpha\, h(1 + n)}{2 \sin^2\alpha}$$

$$= - p'' \frac{2822 \times 4.051}{6 \times 61.25 \times 0.950} + p \frac{0.184 \times 53.125 \times 2.195}{2 \times 0.966}$$

$$= - 32.7\, p'' + 11.1\, p$$

Lower Section (see Fig. 20).—

$$\alpha = 75° 24', \cos \alpha = 0.252, \sin \alpha = 0.968,$$

$$\sin^2 \alpha = 0.937, \sin^3 \alpha = 0.907, h = 166.375,$$

$$h^2 = 27580, n = 0.542, n^2 = 0.294,$$

$$(1 + n) = 1.542, (1 + n + n^2) = 1.836, (n + 2 n^2) = 1.13,$$

$$(n - n^2) = 0.248, r_0 = 51.25.$$

$$V_0 = - 31.4\, p'' + 10.64\, p$$

$$S_0 = + 66.4\, p' - (-14.4\, p''' + 4.89\, p') = + 14.4\, p''' + 61.5\, p'$$

$$S = \frac{p'h(1 + n + n^2)}{3 \sin^2 \alpha} + S_0\, n^2 = p' \frac{166.375 \times 1.836}{3 \times 0.937} + S_0\, n^2$$

$$= 108.6\, p' + (18.1\, p' + 4.23\, p''') = 126.7\, p' + 4.23\, p'''$$

$$
\begin{aligned}
V &= -\frac{p'' h^2 (n + 2n^2)}{6 r_0 \sin^3 \alpha} - \frac{p \cos\alpha \, h(1 + n)}{2 \sin^2 \alpha} - \frac{S_0' (n - n^2)}{\cos\alpha} + V_0 n \\
&= -p'' \frac{27580 \times 1.13}{6 \times 51.25 \times 0.907} - p \frac{0.252 \times 166.4 \times 1.542}{2 \times 0.937} - 0.985 S_0' + 0.542 V_0 \\
&= -111.6 p'' - 34.6 p - (60.7 p'' + 14.2 p'^{V}) + (-17.0 p'' + 5.8 p) \\
&= -28.8 p - 189.3 p'' - 14.2 p'^{V}
\end{aligned}
$$

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3272

STORAGE FOR IRRIGATION WATER IN HUMID AREAS

By T. H. Quackenbush[1]

SYNOPSIS

The factors that affect the development of irrigation storage facilities in humid climates are discussed. Also included are the modifications in design and planning criteria that are necessary in order to properly utilize the engineering knowledge gained on irrigation projects in the western United States.

The irrigated acreage of the 31 eastern states of the United States is estimated to exceed 3,000,000 acres.[2] This is an increase of about 55% during the eight-year period, 1949-1956. This great expansion of the irrigated farmland in the eastern United States has introduced a new and important factor into the planning and development of water supplies.

The line that divides humid from arid lands in the United States is not a sharp one. It is a broad transition zone running across Texas, Oklahoma, Kansas, Nebraska, and North and South Dakota. East of this zone is the humid area which will be discussed in this paper.

In analyzing how irrigation water supply development affects water resources in this area, it should be kept in mind that the water which is used for irrigation is actually consumed, and only a small percentage can be reused for other purposes. This, plus the fact that the maximum irrigation requirements occur during the driest periods when rainfall is least and streams are lowest, certainly emphasizes the need for consideration of the potential irrigation water storage in planning the future development and use of our water resources.

Note.—Published essentially as printed here, in September, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2155. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Irrig. Engr., Soil Conservation Service, Washington, D. C.

[2] "1957 Directory and Buyer's Guide," Irrigation Engineering and Maintenance Magazine, New Orleans, La., August, 1957.

The continued increase in irrigated acreage during both wet and dry years offers evidence that this is more than an "emergency" increase due to unusual weather conditions. Admittedly, much of the monentum that started irrigation farming off to such a rapid growth in the eastern states was due to several severe drought years. However, farming practices and market conditions have so changed that many farmers can no longer afford the risks associated with unfavorable rainfall distribution in any year. Production costs of some crops have increased to such an extent that a further investment in irrigation is warranted to insure maximum high quality production every year.

The average annual rainfall in humid areas would be sufficient to meet the total moisture requirements of most crops if properly distributed throughout the growing season. This ideal rainfall pattern seldom occurs, however, and many crops suffer even in so-called "normal" years. Moreover, runoff from high-intensity storms and deep percolation to below the crop roots also deprive growing plants of a large part of the total rainfall.

The development of irrigation storage in the humid Eastern States up to the present time has been largely limited to the construction of reservoirs by individual farmers. A few group storage projects have been developed and a few more are in the planning stage but in general the constructed storage has been pretty much on an individual farm basis. The 1954 Census of Agriculture shows that 50% of all the farmers irrigating in the humid areas have some type of constructed storage. It is probable that as more and more farmers use irrigation, the unregulated flow of streams and natural lakes will become more fully utilized. As a result farmers will undoubtedly rely, to an increasing extent, on the use of constructed storage facilities.

If this is to be done in an efficient manner, it is essential that the larger group enterprise storage developments also be considered in order to provide the most economical projects.

Future years will undoubtedly see some large irrigation developments installed in the eastern states, but in analyzing the time element involved in large project development, the following factors which differ from conditions in the western states should be considered.

1. The economic feasibility of irrigating some farm crops has not yet been fully established. In planning larger projects which will include diversified farming areas such crops will normally be included and must be considered when computing benefits.
2. The increased profits over nonirrigated farming are much less than in the arid climates. This narrow profit margin further restricts the construction costs that can safely be charged against the land.
3. The rainfall and drought occurrence pattern will often dictate the time of irrigation. This will usually require that all farmers in the area be supplied with the maximum amount of irrigation water at the same time. This will tend to increase costs of the distribution system by requiring greater capacities per acre served.
4. Maintenance costs on canals and ditches will be increased due to the extended periods of non-use, and the more vigorous growth of weedy vegetation due to the higher rainfall. This could be offset with pipelines and lined canals, but again the installation costs would increase.
5. The smaller benefits from irrigation will require that a very high percentage of the land under the distribution system be included as a part of the project in order to provide a favorable cost-benefit ratio. Many farmers are

not ready for irrigation, which makes the formation of an operating organization such as an irrigation district or water users' association more difficult.

6. State laws must be modified to facilitate the formation of irrigation enterprise organizations, and to establish their legal rights to the water. The riparian doctrine which is in effect in the eastern states must be modified in order to protect the large investments that are being made in irrigation works.

Many of these factors do not apply, at least to as great a degree, to the individual farm type of irrigation development. Small watersheds will yield adequate supplies of irrigation water for individual farm acreages. Irrigation water can be used only on specialized crops that produce good returns; extensive distribution systems are not required; the individual farmer has complete control of the system and an operating organization is not needed. At the present time these factors tend to favor the individual type development, which in the long run may or may not be the most economical and efficient type of project.

It is evident that the pattern of growth of irrigation in the East is very similar to what occurred in western United States. The first land to be irrigated in the West was the land that could be supplied with water cheaply and easily. As economic conditions changed and the population increased, more complex and costly projects could be justified.

Similar progress is now evident in the eastern states and at the present time (1959) they are in the first stage of water-supply development which consists of the direct diversions from streams, construction of wells, pumping plants, and storage reservoirs, primarily to serve the needs of the individual farmer.

Whatever the source of water, it must be able to deliver an irrigation stream large enough to cover the irrigated area in the allotted time, or conversely, the size of the irrigated area should be limited to what can be adequately irrigated by the available irrigation stream.

The rate of dependable flow required depends on the size of the area irrigated, the moisture requirements of the crops during periods of peak use, the type of soil in the area to be irrigated, the efficiency of the distribution system and the time allotted to complete one irrigation.

If the proposed water source will not supply irrigation water at a rate that will satisfy needs based on these factors, storage will be required. The amount of storage that is needed may vary from a small reservoir for overnight storage to a reservoir sufficiently large to supply the volume required for the entire irrigation season.

The primary factors that must be considered in determining the volume of storage required for irrigation purposes are as follows; the water requirements of the crop, the farm irrigation efficiencies, the efficiency of the distribution system and the evaporation and seepage losses in the reservoir. The water requirements of the crop, which is the primary factor on which the need for irrigation storage is based, constitutes one of the primary differences between irrigation in the humid eastern states and the arid areas of the west in that the volume of irrigation water required per irrigated acre is somewhat less. This is due primarily to the great difference in rainfall. Harry Blaney[3] shows a typical irrigation water need of 42 in. for alfalfa in the Salt River

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[3] "Consumptive Use and Irrigation Requirements of Crops in Arizona," by Harry F. Blaney and Karl Harris, prepared in cooperation with the Univ. of Arizona, Agric. Experiment sta., December, 1951.

Valley of Arizona while W. H. Dickerson[4] shows an average need of only 10 in. to 14 in. for forage crops in West Virginia, and C. H. M. Van Bavel[5] indicates that the maximum amount of water needed for irrigation in North Carolina for 9 yr out of 10 yr varies from an annual requirement of 8.3 in. to a maximum of 14.4 in., when based on an irrigation application of 2 in. These water needs reflect the actual needs of the plant, and of course, must be modified to allow for application efficiencies, evaporation, seepage, and transmission losses when planning for irrigation water storage requirements. Several procedures have been developed for estimating the water requirements of the crop on which irrigation storage volume must be based. Methods proposed by Blaney and Wayne D. Criddle,[6] H. L. Penman,[7] and C. W. Thornthwaite[8] have been used successfully for estimating potential evapotranspiration or consumptive use.

In the humid areas the "effective" rainfall will often provide a large percentage of this potential evapotranspiration or consumptive use, and therefore must be determined if any of these methods are used to estimate irrigation water storage requirements. This percentage of the rainfall that is effective for the production of farm crops will vary widely depending on the frequency, intensity and total amount of the rainfall occurrence. The method proposed by Van Bavel[5], which recognizes the available moisture-holding capacity of the soil within the root zone, offers a procedure by which irrigation water requirements can be estimated directly, considering the probable frequency and amount of rainfall as it is related to the available moisture capacity of the soil and the rate of use by the crop. This procedure recognizes that the effect of short droughts on crops in the humid area depends in part on the moisture-holding capacity of the soil. A plant growing in a very sandy soil may show early signs of drought, while the same plant in a silt loam may survive a dry period without damage. These differences affect the need for irrigation water in each locality, and must be recognized in the determination of water needs. Van Bavel indicates a maximum seasonal water requirement of 17.6 acre-in. per acre in eastern North Carolina for a soil requiring a 1-in. irrigation application while a soil that will hold an application of four inches has a seasonal requirement of only 10.4 acre-in. per acre.

Another important difference in irrigation water requirements is in the amount of water required for "leaching." The irrigation water used in the Eastern States is generally of high quality and in most cases in the coastal areas where brackish water is used the surplus rainfall seems to adequately handle the leaching of the harmful salts from the root zone. Reservoir capacity, therefore, would not normally be required to store water for leaching purposes.

After the actual irrigation water requirements of the crop have been determined, the farm irrigation efficiency must be estimated. Experience has shown that onfarm irrigation efficiencies in the Eastern States are slightly higher than the average figures quoted for the West based on the following reasons:

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[4] "Rainfall and Rainfall Deficiency in Relation to Requirements for Irrigation in West Virginia," presented at the North Atlantic Sect., ASAE, August 24, 1954.

[5] "Agricultural Drought in North Carolina," by C. H. M. Van Bavel and F. J. Verlinden, Tech. Bulletin No. 122, N. C. Agric. Experiment Sta., June, 1956.

[6] "Determining Water Requirements in Irrigated Areas from Climatological and Irrigation Data," by Harry F. Blaney and Wayne D. Criddle, S. C. S. Tech. Publication 96, U. S. Dept. of Agric., Washington, D. C., August, 1950.

[7] "Natural Evaporation from Open Water, Bare Soil, and Grass," by H. L. Penman, Proceedings, Royal Soc., London, Series A 193, 1948, p. 120.

[8] "An Approach Toward a Rational Classification of Climate," by C. W. Thornthwaite, Geographical Review, Vol. 38, 1948, p. 55.

1. Higher percentage of sprinkler irrigation in the East. The more automatic nature of sprinklers, plus the simplicity of operation, enables the new irrigator to obtain higher efficiencies with less knowledge and effort and the widespread use of pipe distribution systems also decreases transmission losses. As of 1959 78% of all farms irrigating in the eastern states use sprinkler irrigation.

2. Surface irrigation is best adapted to the nearly level soils which have been properly smoothed. A high erosion hazard from rainfall is present on steep land. The low efficiencies that are obtained by surface irrigation on the steep lands of the West should not be a major factor in the East if proper provisions are taken to prevent soil erosion.

Land leveling for surface irrigation has continued a steady growth in the Eastern States. Farmers have found that land properly prepared for surface irrigation provides one of the most efficient surface drainage systems that can be installed on the farm. Land properly leveled facilitates efficient surface irrigation during dry periods and gives adequate surface drainage of surplus rainfall during wet periods.

3. Irrigation water is generally expensive. Nearly all irrigation systems have been installed during the period of higher costs since World War II, and in addition 94% of all farms being irrigated in the East use pumps to supply irrigation water. While total volume requirements are less the cost per acre-inch is often higher than in many areas of the West. This expensive water encourages the average eastern irrigator to be efficient in water use.

4. Smaller applications generally required. The tendency of many crops to develop shallow root systems requires frequent small applications of irrigation water. This tends to reduce losses by overirrigation and deep percolation, recognizing that it also tends to increase the losses by evaporation from the soil surface. The humid area soils generally have fine-textured subsoils at 6 in. to 24 in. deep. The low permeability rates of these subsoils also tends to reduce losses by deep percolation.

Experience in irrigating does not appear to be a primary factor in producing favorable irrigation efficiencies as long as adequate technical assistance is available to the new irrigator. New irrigation farmers in the East seem to accept modern methods and techniques for improving irrigation efficiencies, thus partially offsetting their lack of experience.

When adapting farm irrigation efficiencies obtained in the Western States, these factors should be considered, recognizing, of course, that the design of each irrigation system should be based on the specific site conditions that are encountered.

The evaporation and seepage losses in the reservoir must, of course, also be estimated in the determination of the volume of irrigation storage that is required for any given project. Evaporation losses have been estimated using the methods outlined by M. A. Kohler, T. J. Nordenson and W. E. Fox[9] and Carl Rohwer.[10] These methods have given good results for this purpose to date.

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[9] "Evaporation from Pans and Lakes," by M. A. Kohler, T. J. Nordenson, and W. E. Fox, Research Paper No. 38, U. S. Weather Bur., Dept. of Commerce, Washington, D. C., May, 1955.

Seepage losses should be determined by basing the estimate on the conditions at a specific site. Complete geologic and soild investigations are necessary, and the results of local measurements and research work should be used whenever if is available. Dwight Smith[11] measured combined evaporation and seepage losses of 52.84 in. and 49.44 in. in 1953 and 1954 at McCredie, Mo. Data of this type would be of great value in designing storage reservoirs where similar conditions are encountered. Additional measurements of seepage and evaporation under many varying conditions are urgently needed in order to more accurately determine these expected losses, especially as they apply to the smaller farm reservoir. Smith also points out the importance of steep side slopes and increased depths in the reservoir area as a factor in reducing evaporation and seepage losses. Plastics, butyl rubber, asphalt, soil cement, and many other materials are now being utilized to reduce reservoir seepage losses. It should be emphasized that complete investigations prior to construction and the use of alternative reservoir sites can often eliminate the need for expensive reservoir linings.

One of the most difficult jobs in designing small irrigation storage reservoirs in the humid areas is the determination of the expected water yields from the watershed above the reservoir. Fortunately, some stream-gaging records are available, on which estimates of watershed yields can be based. H. O. Ogrosky[12] points out that there are approximately 2,962 stream-gaging records available for drainage areas less than 100 sq miles. The Agricultural Research Service, United States Department of Agriculture, has obtained records on 328 watersheds of less than 10 sq miles, with 294 of them less than one square mile in area. This data has been assembled by the Agricultural Research Service[13] and provides some useful data for estimating expected water yields from small watersheds.

The steady growth of irrigation in the eastern states emphasizes the need for careful planning and good engineering in designing irrigation storage reservoirs. Additional research and investigations on various irrigation problems should be encouraged to insure that our limited water supplies are fully and efficiently utilized.

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[10] "Evaporation from Free Water Surfaces," by Carl Rohwer, Tech. Bulletin No. 271, U. S. Dept. of Agric., Washington, D. C., December, 1951.

[11] "Storage and Pond Design," by Dwight D. Smith, Agricultural Engineering, Vol. 36, No. 11, November, 1955.

[12] "A New Look at Small Watershed Hydrology," by H. O. Ogrosky, Hydraulic Engineer, Soil Conservation Service, Washington, D. C.

[13] "Monthly Precipitation and Runoff for Small Agricultural Watersheds in the United States," U. S. D. A., Agric. Research Service, Soil and Water Conservation Branch, Washington, D. C., 1957.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3203

TRITIUM AS A GROUND WATER TRACER

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SYNOPSIS

Tritium in the form of tritiated water is a nearly perfect ground water tracer. Its only serious disadvantage is derived from its radioactive properties and the potential effects on public health of extremely small concentrations of radioisotopes. The potential biological risks of using tritium in large scale water tracing operations are treated and compared with the possible benefits.

INTRODUCTION

The ability to label and subsequently identify water, even after a movement of several miles through the earth, is of considerable interest to the sanitary engineer and others concerned with water resource protection or development. Perhaps most typical of tracer applications is the relatively simple situation involving the origin of contamination in a well water and whether a nearby leaching field was the origin. On a larger scale, there is often need for detailed information on the the rate and direction of ground water movement in order to evaluate the ultimate capacity of a ground water resource. Typical of situations requiring knowledge of ground water flow are the sewage reclamation operations now under development in the Los Angeles area. Questions relating to the maximum possible rate of pollution travel, who is benefiting from the recharge operation, and how much water is ultimately recovered can

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Note.—Published essentially as printed here, in November, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2660. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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be more correctly and economically answered with the aid of tracers. One of the most potentially interesting applications of water tracers is for geological exploration of deep formations of the earth in connection with injection disposal of radiactive and other toxic wastes.

There are numerous substances that will serve satisfactorily as water tracers, but only under a certain set of circumstances. In setting forth the desirable characteristics of the ideal or perfect tracer, it immediately becomes apparent that very few, if any, materials are entirely satisfactory. What are the criteria of the ideal ground water tracer? Obviously, it must move in exactly the same manner as the water that it is expected to trace without in any way modifying the transmission properties of the aquifer. Moreover, it must obviously be detectable. A most important criterion is low adsorptive loss to the porous medium, a condition that immediately eliminates all cations, including both the normal chemical forms and those available as radioisotopes. The chloride ion and other members of the halogen group, such as iodine, are adsorbed relatively little and have been used successfully in many instances. For example, iodine 131, accompanied by stable iodine as a carrier, has found application in the secondary recovery of petroleum. The organic dyes, particularly sodium fluorescein, have been widely used as water tracers, but have been repeatedly demonstrated to be unreliable where the test formation contained humus or other organic material. For qualitative answers fluorescein will continue to serve usefully in ground water tracing.

As a general rule, small concentrations of water-miscible substances are likely to be retained, at least to some degree, by adsorption on soil colloids. For this reason a majority of the radionuclides, unless "carried" by large concentrations of a stable isotope, are unsuitable as water tracers. It is apparent that the most likely tracer of water is water itself. Ordinary water contains three isotopes of hydrogen; hydrogen 2 or deuterium and hydrogen 3 or tritium, and of course the most common, hydrogen of mass number one. In addition, there are three naturally occurring isotopes of oxygen as well as several artificially created radioisotopes. These isotopes, when a part of the water molecule, will move through most natural media without significant adsorptive loss. From the standpoint of faithfully tracing water, they would appear to be ideal.

In addition to the adsorption criterion, a tracer should be measureable at a reasonable cost in low concentrations. It should cause no impairment of any beneficial use of the labeled water, and should undergo no physical or chemical change that would either hinder its measurement or result in its loss from the water mass. The cost of the tracer, together with the cost of measurement, is often an important factor, particularly where a study may require several hundred analyses and entail the labeling of hundreds of millions of gallons of water. It is generally preferable that the tracer be nearly absent from the water to be labeled, although there are instances in which this is not necessary; the investigator merely relies on concentration differences to distinguish between the two waters.

The tracer that appears to meet these many requirements most completely is the hydrogen isotope tritium. Tritium is to be preferred over the other components of water, such as deuterium or the oxygen isotopes, primarily on the basis of either the cost of the isotope or its measurement at the desired level of sensitivity.

NATURE AND OCCURRENCE OF TRITIUM

Tritium is the only radioisotope of hydrogen. It has a mass number of 3 and decays by pure beta emission with a half-life of 12.5 yr. Tritium occurs naturally, being produced at a constant rate in the earth's atmosphere as a result of cosmic ray interaction with nitrogen atoms. Libby[2,3] has reported the average natural world-wide tritium content of land rainfall as 3.3 tritium atoms per 10^{18} hydrogen atoms, a radioactivity concentration equal to 10^{-8} microcurie (μc) per cubic centimeter. A microcurie is the amount of radioactive material undergoing 2,200,000 disintegrations per minute. Occasionally Libby observed precipitation with a natural tritium content exceeding 2×10^{-7} μc/ml.

In the spring of 1954, the entire hydrologic distribution of the world's tritium was suddenly altered by the Castle series of thermonuclear weapons tested in the Pacific. As a result of this test series, the concentration of tritium in precipitation in North America rose from about 10^{-8} μc/ml to more than 8×10^{-6} μc/ml, a nearly one thousand fold increase. By mid-1955, the tritium content of precipitation in the Northern Hemisphere had decreased to 10^{-7} μc/ml, approximately ten times the pre-weapons test figure. In 1956, tritium concentrations rose a second time and in the past two years have been fluctuating between ten and one hundred times the natural value of 10^{-8} μc/ml; these fluctuations are presumably due to the continued testing of thermonuclear weapons and to variations in transport and deposition of tritium from the upper atmosphere. Measurements made in the spring of 1959, both in Europe and in the United States, indicated rainfall concentrations of tritium were again exceeding the 3×10^{-6} μc/ml level.

The Castle series of weapons tests introduced into the atmosphere, and subsequently into the earth's surface and ground waters, an enormous quantity of tritium labeled water. Several investigators, including Begemann and Libby[4], have measured the concentration of this tracer in precipitation and surface and ground waters, and have arrived at general conclusions regarding the gross movement of water in the atmosphere and on earth. Studies of movement of bomb produced tritium through an aquifer in New Mexico have been reported by von Buttlar[5]. The New Mexico study suggests that a "post-Castle" tritium content of precipitation may provide, without cost to the investigator, a useful hydrologic tool. In certain instances this may prove to be the case. However, the occurrence and movement of tritium in ground and surface waters

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[2] "Research to Assay Rain and Surface Water for Natural Tritium Content," by W. F. Libby, Final Report, Air Force Contract A.F. 18, 600-564, Univ. of Chicago, June 1, 1954.

[3] "The Natural Distribution of Tritium," by Kaufman, Sheldon and W. F. Libby, Physical Review, 93, 6, pp. 1337-1344, March 15, 1954.

[4] "Continental Water Balance, Ground Water Inventory and Storage Times, Surface Ocean Mixing Rates and World-Wide Water Circulations Patterns from Cosmic-Ray and Bomb Tritium," by Friedrich Begemann and W. F. Libby, Geochemica et Cosmochimica Acta, Vol. 12, No. 4, pp. 277-296, 1947.

[5] "Investigating Ground Water by Analysis of Atmospheric Tritium," by Har von Buttlar, Journal, A. W. W. A., 50, 11, pp. 1533-1538, November, 1958.

has been so greatly confused by subsequent weapons tests that it is doubtful whether it will prove of significant value.

MEASUREMENT OF TRITIUM

Tritium is a low energy beta emitter, the maximum energy being only 0.018 mev and the average energy 0.0059 mev. For this reason, and because the tritium content of a sample cannot be raised by ordinary concentration techniques, tritium measurement represents a rather difficult problem. Two measurement methods are available. Tritium, in the form of tritiated water, may be converted into the molecular form of hydrogen gas and introduced into one of the conventional types of gas-filled detectors, such as a Geiger or proportional counter. Under these conditions, the disadvantages of the low decay energy are largely eliminated, and the measurement made with nearly 100% efficiency.

A second and more recently developed method of tritium measurement uses the liquid scintillation counter. With this equipment, a tritiated water sample is dissolved in a transparent solvent containing a phosphor and placed in a glass vial. The low energy beta radiation (of low penetrating power) interacts with the phosphor to produce small pulses of light that readily pass through the solvent-water mixture and the glass container. The sample is viewed by two photo-multiplier tubes, often referred to as photoelectric cells, that convert the light to electrical pulses. The electrical pulses are passed through electronic circuitry to a scaler or recorder. Although the liquid scintillation counter is complex and costly, the sample preparation and counting procedure are quite simple and require little technician time or experience in execution.

The sensitivities of the gas counting and liquid scintillation counting systems are essentially comparable and will permit the detection of tritium concentrations as low as 5×10^{-7} $\mu c/ml$ with a counting period of 30 minutes.[6] Unfortunately, this "sensitivity" is not adquate to detect natural tritium levels or even to measure with precision those resulting from weapons testing. This inadequacy may be overcome by electrolyzing a liter or more of water, a process that reduces the volume of the sample, but also increases greatly the tritium concentration of the remaining water. The electrolysis-gas counting methodology is currently being used by several groups in the United States. A measurement by this technique requires several weeks to complete and is estimated to cost on the order of $200.

The application of natural tritium obviously has several serious disadvantages. As noted above, its measurement is time-consuming and costly. Furthermore, as considered in the previous section, the investigator has no control over its occurrence that has become increasingly complex with the continued testing of thermonuclear weapons.

PUBLIC HEALTH IMPLICATIONS OF USING RADIOACTIVE TRACERS

The precise extent of biological damage to humans from exposure to low-levels of ionizing radiations is yet uncertain. However, it is generally agreed that all radiation exposure, even that of natural background, is probably harmful to man and his descendants. An increasing mass of experimental data lends support to the often expressed philosophy that all radiation exposure is harmful, and hence, all unnecessary exposure to radiation should be avoided. Unfortunately, an equally convincing mass of data leads to the conclusion that any operation involving radioactive materials must necessarily result in some release of these materials to the biosphere. Thus, the engineer-physicist group concerned with developing the material benefits of nuclear science is brought into sharp conflict with the physician-biologist group concerned primarily with the protection of individual and public health. The public, at best only partially informed, may be expected to view radiation effects on a personal basis and consequently support the position of the biologist. How may this apparent impasse be resolved to the mutual safety and benefit of all concerned? The only answer would appear to be the equating of material benefits of nuclear science to the costs measured in terms of undesirable biological effects. Furthermore, it is essential that "costs," or individual "risks" be measured in the same framework as other risks of everyday life, that is, the risks of driving, smoking, over-eating, breathing polluted atmospheres, and perhaps even crossing the street.

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[6] "Low-Level Tritium Measurement with the Liquid Scintillation Spectrometer," by R. M. Hours and W. J. Kaufman, Paper No. 51, Nuclear Engrg. and Science Conf., April, 1960.

Prior to 1934, the acceptable occupational exposure level to radiation was 100 rem per yr. The occupational level has been progressively reduced and at present rests at 5 rem per yr, as recommended by the International Commission on Radiological Protection in April, 1956. (The rem is the quantity of any ionizing radiation such that the energy imparted to a biological system per gram of living matter has the same biological effect as that from x-radiation giving an absorbed dose of 100 ergs per gram.) The reductions represent compromises between zero and the capabilities of technology both to use and to control radiation. It is significant that at none of these levels has there ever been demonstrated damage to the individual. The International Commission now recommends that the whole population dose up to age 30 not exceed 10 rem considering all sources of radiation exposure except natural background. Subtracting from this value exposures resulting from medical usage of radiation and fallout, a dose of 4 to 5 rem for 30 yr, remains that might be looked upon as a "bank" from which withdrawals may be made (providing they are invested in enterprises bringing real and significant benefits to the affected population).

An examination of the most recent recommendations of the United States National Committee on Radiation Protection and Measurements (N.C.R.P.) has been published by L. S. Taylor[7]. This Committee, on which there are representatives from the AMA, the U. S. Public Health Service, and other public and private agencies, has recommended that the radiation or radioactive material outside a controlled area, but attributable to normal operations in that area, shall be such that it is improbable that any individual will receive a radiation dose of more than 0.5 rem in any one year. This value is 10% of the occupational limit for whole body or gonadal exposure. Similarly, the National Committee has recommended that concentrations of radioisotopes in air and water outside a controlled area should not exceed one-tenth of the maximum permissible concentrations for occupational exposure. In each case, the concentrations may be averaged over a period of one year. It should be noted that the National Committee's value of 0.5 rem per year for 30 yr amounts to 15 rem, exceeding by a factor of three the International Commission's exposure bank of 4 to 5 rem. This is not an inconsistency, since 0.5

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[7] "Maximum Permissible Radiation Exposures to Man," by L. S. Taylor, Health Physics, Vol. 1, No. 2, pp. 200-204, September, 1958.

rem per yr is the individual maximum that is most likely to be reached for any large group of people, whereas, the 4 to 5 rem value is the average exposure for an entire population group.

The deliberations leading to the previously cited recommendations were largely concerned with genetic damage, it having been assumed that a threshold radiation dose existed below which somatic damage did not occur. There is, however, convincing evidence, from studies of animal populations, that whole body radiation exposures ranging from 100 to 1200 roentgens result in shortening of life-span in direct linear proportion to the absorbed dose. This relationship has been shown to exist for laboratory animals with both acute and chronic exposures. If the linear relationship of radiation dose to effect can be properly applied by exprapolation to very low doses, the implications are of great interest to the sanitary engineer. Jones[8] concludes that small-animal data suggest that one roentgen of radiation exposure is equivalent to 5 to 15 days of physiologic aging, that is, life-span shortening. He further indicates this relation to be valid for doses of a few roentgens or less. As might be expected, all biophysicists do not concur with the low-dose, proportional-effect concept, pointing to evidence suggesting that small radiation doses may stimulate physiologic mechanisms that appear to extend the average life-span, perhaps by affording the animal additional protection against disease. It is evident from the literature that a great deal of additional research will be necessary to demonstrate conclusively the validity of the proportional effect concept at low radiation dosages. It is improbable that this will ever be accomplished on an ipidemiological basis. However, existing experimental evidence and scientific opinion favoring the proportional concept will most certainly influence public health policy in establishing standards not regulating radiation exposure of the population. It would thus appear that we are likely to pursue a conservative investment policy in making withdrawals from our exposure "bank", and it is quite likely that many public health workers will even find it most difficult to accept the bank philosophy.

In the previous paragraph, somatic effects expressed as life-span shortening of the presently living population were considered. To realize the entire consequence of radiation exposure, a brief analysis of the genetic effects of radiation is in order. The best current estimates of the genetic effect of radiation place the mammalian mutation-doubling dose at about 50 r with most estimates[9] ranging from 30 to 80 r. Since natural background ranges from 4 to 5 r, it is apparent that only about 10% of the normal human mutation rate is due to radiation and that the "bank" suggested by the International Commission, if fully and continuously used for the development of nuclear technology, would ultimately result in approximately a 10% increase in the human burden of mutant genes. What is the cost of this additional burden? From the National Academy of Science report it can be computed that increasing human radiation exposure by 5 r per generation would result, after perhaps 40 generations, in 0.2% of live births having tangible genetic defects.

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[8] "The Nature of Radioactive Fallout and its Effects on Man," by H. B. Jones, Hearing before the Special Subcommittee on Radiation, Eighty-Fifth Congress, Washington, 1957, pp. 1100-1137.

[9] "The Biological Effects of Atomic Radiation," Natl. Academy of Sciences, Summary Report, 1956.

MAXIMUM PERMISSIBLE CONCENTRATIONS OF TRITIUM IN WATER

Tritium, introduced into the human body in the form of tritiated water, is very quickly dispersed in the body fluids. Pinson and Langham[10] have measured the period of equilibration of ingested tritiated water with body fluids to be from 2 to 2.5 hr. If no further intake of tritium occurs, a period of approximately 12 days is necessary for one-half of the tritium burden to be excreted from the body. This period is termed the "biological half-life." Since the radioactive half-life is considerably greater (12.5 yr), the biological half-life essentially equals the effective half-life that determines the rate of loss of a radionuclide from an organ.

The 1957 U.S. Atomic Energy Commission regulations for protection against radiation[11] place the Maximum Permissible Concentration of tritium in water at 0.016 μc/ml, this value applying to the general population in the vicinity of a controlled area. (The Maximum Permissible Concentration (M.P.C.) of tritium in water for continuous occupational exposure, stated elsewhere[12] (June 5, 1959), is 0.03 μc/cc. One tenth of this value may be regarded as maximum permissible in the neighborhood of a controlled area or nuclear installation.) Using the more recent recommendations of the National Committee on Radiation Protection of 0.5 rem per year and other information on the biological effects of tritium radiation, a maximum, permissible concentration of 0.003 μc/ml may be computed. If an individual consumed water containing this concentration of tritium for several weeks, the tritium body-burden would approach 120 μc and the whole body exposure rate would reach the permissible limit of 0.010 rem per week or 0.5 rem per year. In the case of short duration or intermittant exposure, it is permissible to compute the exposure rate on a yearly average basis. It is worthy to note that the computation of radiation exposures to the whole body or gonads from tritium is probably one of the most reliable of all the internal dose computations.

SELECTING A TRITIUM CONCENTRATION FOR WATER TRACING

Let us assume that we are to trace a body of ground water that serves as a source of supply for a community of several hundred thousand persons, and that tritium is the tracer under consideration. What concentration should be used, or in fact should we permit any tritium radioactivity to be employed? On the basis of the U.S. Atomic Energy Commission (AEC) regulations, derived from the recommendations of the National Committee on Radiation Protection, we could presumably use some fraction of the Maximum Permissible Concentration for the population at large. Since the duration of the investigation would perhaps be not more than a few years, no individual would be exposed for more than a fraction of a lifetime. Thus, considering the official M.P.C., the relatively short duration of exposure, the sensitivity of our measuring equipment, and the cost of tritium, we might arrive at a tracer concentration of about 50% of the lifetime M.P.C. The AEC has, in fact, granted a license for the use of this concentration for tracing a domestic water supply.

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[10] Journal of Applied Physiology, by E. A. Pinson and W. H. Langham, Vol. 10, No. 1, pp. 108-126, January, 1957.

[11] "Standards for Protection Against Radiation," Federal Register, Title 10, Chapter 1, Part 20, January 29, 1957.

[12] Natl. Bur. of Standards Handbook, Vol. 69, June 5, 1959.

Let us examine the project in terms of the material benefits likely to accrue and objectively compare these benefits to the biological risks of the investigation. For this comparison it is convenient to take an actual proposed investigation that is now under consideration.

The Los Angeles County Flood Control District has for some years followed a practice of replenishing the ground water at some sixteen locations in the county by artificially introducing surplus water. In such operations, the earth serves as both a storage reservoir and distribution system for the communities and industries utilizing the affected aquifers. The spreading basin located at the mouth of the San Gabriel Canyon is typical of such installations. Water is a costly and essential commodity in the life and economy of the San Gabriel Valley and the coastal regions of Los Angeles county. In purchasing water and discharging it into the earth, several questions are immediately raised. The most important of these is what fraction of the water is actually recovered for beneficial use. Another question of some importance in financing an expanded replenishment program is who are the beneficients and to what extent are various pumpers receiving the spread waters. Partial answers to these questions can, of course, be obtained by an examination of pumping records, ground water levels, and similar hydrologic data. However, it appears that only by labeling the recharged water can reliable quantitative data be obtained. What are these answers worth in dollars? This is a most difficult question, and the answer can only be arrived at in an indirect manner. The combined ultimate water use of the Main San Gabriel Basin and the East Central Coastal Basin is estimated to be 445,000 acre ft annually. Placing the value of water at \$30 per acre ft, the annual water "bill" will amount ultimately to nearly \$15,000,000. With only local water available to meet this demand, an annual over-draft of 163,000 acre ft is predicted. This demand can, to a large measure, be feasibly met by replenishment operations. Such operations would make ground water recharge a 3 million dollar a year public enterprise. Knowledge enabling the efficient operation of the replenishment system could, over a 20-yr period, conceivably benefit the population to the extent of several millions of dollars.

It has been proposed to introduce tritium into a flow of 40 cfs such that the tritium concentration amounts to 1.6×10^{-4} μ c/ml. This water would be introduced into spreading basins at the mouth of the San Gabriel Canyon and would ultimately find its way into the main San Gabriel Basin and perhaps even through the Whittier Narrows to the Coastal Basin. Under the conditions of the test, the tracer would be added continuously for 35 days, labeling a total of 825 million gal of water and requiring 500 curies of tritium. Samples would then be taken from various wells serving the major municipalities of the region and the movement of the traced water mass followed to the limits of measurement sensitivity. It is quite possible that the traced water would be detected through the Main Basin and perhaps even to the Pacific Ocean. It should be noted that the tritium detection limit of 5×10^{-7} μc/ml noted earlier will enable the detection of the labeled water even after a 300-fold dilution.

What are the potential biological risks of such an investigation? In estimating these risks it is, of course, essential to anticipate the worst possible conditions that could develop. However, the assumptions made in estimating the effects of the worst condition should be plausible and guided by established scientific principles. Where a reasonable doubt exists we should use the conservative assumption in the interests of erring on the side of safety. Let us consider two possible situations involving the exposure of individuals and populations:

1. Assume that the entire 825 million gal remains in the locale of the spreading basin and serves a small group of individuals throughout their entire lives. This is a highly improbable assumption considering the mobility of ground water. It is examined solely to illustrate the maximum possible effects on a single individual. The ingestion of 1.6×10^{-4} μc/ml would cause an exposure over a period of 70 yr of 1.9 rem. Adopting the proportional effect philosophy and using Jones' value of 10 days of life-span shortening per roentgen (or 6 days per rem), the statistical life-span reduction would be about 11 days.

A more probable maximum individual effect would be reached by assuming a more reasonable, yet conservative, exposure period; 1 yr for example, at the full concentration of 1.6×10^{-4} μc/ml. The effect on life-span would be reduced by 1/70 or to about 4 hr.

2. Assume that the entire 825 million gal are used by only domestic consumers of the Main and Coastal Basins over a period of several years. Here we are obliged to make three additional assumptions, a daily per capita water consumption of 150 gal, a reuse factor of 2, the latter resulting from a natural and artificial recharge of municipal sewage, and a fractional water use by domestic consumers of 0.5. If it is assumed that the period of use is small in comparison to the physical half-life of tritium, we may omit the reduction in effect due to radioactive decay. The total population dose may then be computed as, (total individual dose) x (total number of persons exposed), or total population dose = 126 (tracer used, in curies) x (fraction of water used by domestic consumers) x (reuse factor) ÷ (per capita water use in gal per day) $= \frac{126 \times 500 \times 0.5 \times 2}{150} = 420$ rem - pop. (The rem is related to the radiation adsorbed dose (rad) by the relative biological effectiveness of the radiation. In the case of tritium, this quantity is 1.7 and hence: rem = 1.7 x rad.) The value 420 rem-pop. is independent of any dilution of the tracer, of the number of persons consuming the traced water, and of the period of consumption. Again assuming a shortening of life-span of 6 days per rem, a total life-span cost to the present generation of consumers of 2,500 man days results. This would appear to be an excessively large value, but spread over a population of 100,000, it would amount to an individual cost of only 0.6 hr.

It is of interest to take a specific disease and compute its particular contribution to the total life-span shortening effect. Lewis[13] has estimated the population mortality rate of radiation induced leukemia to be 0.2 per 100,000 per yr for each roentgen of radiation exposure. Muller[14] has computed that one absorbed roentgen of whole-body radiation would cause about 6 cases of leukemia per 100,000 exposed individuals. In the case of the proposed Los Angeles County investigation, there could result 250 man-$\text{rad} \times \dfrac{\text{6 leukemia cases per rad}}{\text{100,000 persons}}$ or 0.017 cases of leukemia. Since the incidence of leukemia is approximately 5 cases per 100,000 per year, it is obvious that the occurrence of 0.017 cases during and subsequent to the test period would hardly be significant and certainly not statistically distinguishable as a deviation from the normal rate.

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[13] "Leukemia and Ionizing Radiations," by E. B. Lewis, Science, Vol. 125, pp. 965-972, May 17, 1957.

[14] H. J. Muller, Amer. Journal of Human Genetics, Vol. 2, 1950, p. 11.

To what extent will the proposed test increase the genetic burden of the exposed population? Assuming 50% of the population in the Main and Coastal basins are of age 30 or less, the population exposure of genetic significance would be 250/2 or 125 rad-population. From the Natural Academy of Sciences Report[15] it may be estimated that the 125 roentgen-pop. would introduce 0.4 mutants. Assuming an average normal population burden of 5 mutant genes, the 0.4 mutants spread over a population of 50,000 persons would only increase the mutant burden by 0.00017%.

How do the biologic effects of the proposed study compare with the effects of common environmental factors that modify health or cause early death? Jones[16] has computed the life shortening effects associated with the normal characteristics and habits of man. These effects are expressed as years loss of life-span. Several typical values are as follows:

City versus country dwelling	-5 yr
Smoking 1 package of cigarettes per day	-9 yr
25% overweight	-3.6 yr
Accidental death to an individual in U.S.A.	-2.3 yr
Pedestrian motor-vehicle accidents for an individual living in an urban area	-0.4 yr
Motor-vehicle accidents involving driver and passengers	-0.67 yr

In comparison with the preceding figures, the 2,500 man-days lost per assumed 100,000 persons affected (or 0.00007 yr per individual) would certainly appear to be of little significance. What are the total consequences of the 0.4 undesirable mutant genes introduced into the population? It has been estimated that the present mutant burden of the human race results in a life-span shortening of as much as 10 yr. If we assume the mutant burden to be 5 genes, one gene contributes, on the average, 2 yr of life-span shortening to any individual carrying it. If we further assume an elimination rate of the induced mutant of 5% per generation and a static population, it can be shown that the 0.4 genes might possibly cause a total life-span shortening in the next 40 generations of 0.4 x 2 x 20 = 16 years of 5900 man-days. It might be stated that the cost in genetic life-span shortening per living individual to his descendants is about 0.0017 yr.

The total biological expenditure, both somatic and genetic, might be on the order of 2,500 + 5,900 or 8,400 man-days, whereas the potential material benefits of the investigative venture might well range over one million dollars.

The argument is often presented that most risks of living, such as those cited in the preceding list, are incurred voluntarily by each individual, whereas introducing radioactivity into the environment exposes all persons to the risk of biological damage without the individual's consent. To a degree this is certainly a valid contention. However, it is difficult to imagine how one could "voluntarily" avoid being a pedestrian or, in a city such as Los Angeles, avoid being an automobile passenger.

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[15] "The Biological Effects of Atomic Radiation," Natl. Academy of Science, Summary Report, 1956, p. 27.

[16] "Estimation of Effect of Radiation Upon Human Health and Life Span," by H. B. Jones, Proceedings, Health Physics Society, pp. 114-126, June, 1956.

CONCLUSIONS

Tritium, in the chemical form of tritiated water, is a nearly ideal water tracer and perhaps the only satisfactory means of tracing water great distance through the earth. The methods available for measuring tritium are sufficiently sensitive to permit the use of concentrations only 1% of the Maximum Permissible Concentration cited in U. S. AEC regulations. However, the belief that physiologic aging and the incidence of leukemia are increased in direct linear proportion to even very small radiation doses, raises serious doubt regarding the propriety of tritium as a tracer in which human consumption of the traced water occurs. It is believed that the use of tritium is justified if it can be demonstrated that the material benefits of its use are commensurate to the total biological risks to the affected population. The case of a proposed tracer study in Los Angeles County is cited as a situation in which the use of tritium would be justified.

In recent years ionizing radiation has been singled out among the host of stresses endured by the human race and given exhaustive scrutiny. This emphasis on the effects of radiation, though certainly not to be belittled, is a two-edged sword. It has alerted the public health worker to the hazards of radiation, but it has also made it nearly impossible to consider radiation hazards in the same framework as the innumerable other hazards to longevity that we encounter each day of life. We are unable to equate objectively biologic risk and material benefit, yet we must. It is a simple fact that the human race must pay a price in biologic damage if it is to realize any significant benefit from nuclear science. The sanitary engineer can play an important role in radiation control, providing he realistically appraises benefit and risk and can convince the public his equation is a valid and acceptable one.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3205

ACTIVATED CARBON REMOVAL OF HYDROGEN SULFIDE

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SYNOPSIS

The removal of hydrogen sulfide from an air-gas mixture presents many problems. The paper deals with the removal by one media only (activated carbon), but considers the many types of conditions that could occur, as well as the basic conditions that are created during the dynamic chemisorption.

INTRODUCTION

The problem of removing certain gases from an air-gas mixture, or the prevention of the gases from reaching the atmosphere, is becoming a more and more important problem with continued urbanization. The problem is not confined to any one industry or city, but may be found in nearly any location in which dense population exists. Gases that are nuisances, obnoxious, explosive, or toxic are the main causes of the increasing difficulties in the fields of air pollution and industrial hygiene.

There are many ways in which a gas can be eliminated from the atmosphere or from other gases; among them are adsorption, absorption, chemical reactions, and physical processes. Some of the methods have many applications, while others have a limited number of uses.

This paper is concerned with the evaluation of only one gas, hydrogen sulfide mixed with air, being removed by one media, activated carbon. In addition,

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Note.—Published essentially as printed here, in November, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2641. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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the study is basically one relating to the physical conditions that occur rather than the chemical reactions that might take place in the process.

Since hydrogen sulfide is extremely obnoxious (detectable by the nose at a concentration of 0.1 ppm) at low concentrations, and very toxic at high concentrations (maximum allowable concentration for an 8-hr period is 20 ppm), it is imperative that little or none of it reach the atmosphere that must be used for the life processes of humans, animals, or plants.

Dynamic tests were performed using three different carbons in order to determine the type of action occurring and to obtain a comparison of carbons made under varied conditions using different substances in its manufacture. These tests were run at different flow rates, filter sizes, air temperatures, humidities, and hydrogen sulfide concentrations to determine those factors that are the most important in the resulting removal process.

The removal of hydrogen sulfide from an air-hydrogen sulfide mixture was found to be a chemical reaction rather than an adsorption process. Three factors substantiate the above conclusion. First, as the precentage of hydrogen sulfide in the total flow reached a certain level, the carbon became hot and glowed, presumably from the heat liberated due to oxidation of the hydrogen sulfide. Second, it was found that high humidities and high temperatures tend to increase hydrogen sulfide removal, contrary to adsorption theories. Third, as the total flow rate increased the carbon would remove less hydrogen sulfide, thus exhibiting the time element typical of a chemical reaction.

Although the research shows that the carbon could be of considerable value in the removal of hydrogen sulfide from such structures as large sanitary sewers and emission stacks, in which the concentration of the gas is low, it is felt that more comprehensive research should be performed before any extensive or large gas concentration application is attempted. Such factors as other carbons, non-carbon removal material, more precise instrumentation, and somewhat larger scale units are some of the variables in the problem that should be investigated.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, and are arranged alphabetically, for convenience of reference, in the Appendix.

History of Activated Carbon.—Activated carbon is an outgrowth of the use of charcoal and bone char. The earliest recognized date for the use of charcoal as a purification medium is 2,000 B.C., at which time the Egyptians filtered their water through it to remove impurities. The use of charcoal in the field of medicine has been known since 1,550 B.C. The first time the use of charcoal was recognized as a material capable of adsorbing gases was in 1773. In 1854, charcoal was suggested for use in the ventilation of sewers and described in a manner similar to that used in present day gas masks. However, it was not until World War I that activated charcoal came into being; in Europe it was used in gas masks when the German army began gas warfare. After receiving a great deal of publicity during and after World War I, activated carbon was thought to be a cure-all for a number of applications, and, after the war, a number of uses were tried for its application in industry.

Manufacture of Activated Carbon.—Over 1,000 patents are on file covering the manufacture of activated carbon. Undoubtedly, there are many times that number of different processes in existence. Basically, the material is first carbonized at elevated temperatures, allowing the distilled gases to escape from the material and leaving a very porous carbon material. Additional steps are also used, as is evident from the number of patents, but most of these are secondary processes.

Adsorption Theory.—"Adsorption" is defined as an accumulation or concentration of a substance at a surface or interface. "Absorption" is defined as a deeper penetration in which one substance surrounds or engulfs the other substance, usually in the presence of a chemical reaction. It has been found that in many cases there is much doubt as to which of these actions is present, and, in most cases of what is referred to as adsorption, since some of the material is actually being absorbed. The term "sorption" has been suggested as a term to use regardless of which of the two actions is present or predominant.

Most theoretical work on the adsorption of gases on activated carbon is done under static conditions. This type of adsorption is carried out in a closed chamber and held at constant temperature in which the carbon is placed in the presence of the gas that is to be adsorbed. A manometer is placed in the chamber so that it can be read from the outside, and the pressure in the chamber can be noted at all times. The carbon, as it adsorbs the gas, will cause the pressure in the chamber to be reduced as the gas is adsorbed by the carbon. When adsorption ceases to take place, the partial pressure in the chamber reaches a static condition. If this static pressure, for several runs, is plotted against the cubic centimeters of gas adsorbed per gram of carbon, a straight line will be approximated on log-log paper. This line indicates that the relationship is an exponential function. The resulting equation is known as an isotherm. It might be mentioned that nearly all isotherms using activated carbon in static adsorption have positive slopes, indicating that the material is a better adsorber at greater than atmospheric pressures.

"Surface condensation" was the name first used to explain the phenomena of adsorption. This name explains quite well one type of adsorption that occurs frequently and that is now known as physical adsorption. In this type of adsorption, the gas molecules have a greater affinity for the solid surface of the carbon that they are to be adsorbed upon than they do for other molecules of the same gas. Because of this action, they will cling to the surface of the carbon rather than follow the stream path through the carbon. Even though there is a small amount of heat liberated, surface condensation is purely physical in nature.

Chemical adsorption, or "chemisorption," is also considered to be as important in the adsorption process as the physical adsorption previously mentioned. Many gases have been adsorbed on carbon, and when they are driven off, they come off as a compound of the gas and the carbon. A classical example of this type of adsorption is the adsorption of oxygen on carbon. If the oxygen is adsorbed at a relatively low temperature, it can be removed by elevating the temperature of the carbon. The gas that comes off the carbon, however, is not oxygen, but carbon monoxide, thus indicating there is a stronger bond present in the carbon than would be expected if only physical forces were present.

Chemisorption is usually accompanied by higher temperatures than pure physical adsorption, and most gases are found to be adsorbed to a greater degree by this method than by the physical adsorption.

In most actual cases, both types of adsorption take place at the same time, with the temperature rise of the removal media determining which of the two is the predominent action at any instant. Hereafter, in this paper, the term "sorption" will be used regardless of whether the process is physical, chemical, or both.

The sorption of one gas is known to be retarded by the presence of other gases even if the other gases are only slightly sorbable. Since the gas that is to be sorbed mixes with the other gases present in the system, it is more difficult for a molecule of the gas to be sorbed to reach the carbon surface; this is due to the volume relationship of the gas to be sorbed with the total volume of gases passing through the filter.

The basic properties of different activated carbons are usually somewhat different. Therefore, a carbon with one property might be a very good sorbing material for a particular gas, but because of this same property, this carbon might be a very poor sorber of some other gas. This difference can be attributed to the material from which the carbon is made, as well as the process used in its manufacture; the results of this research clearly substantiate this fact.

Activated Carbon Properties.—Activated carbon has an extremely high surface area. Carbons have been reported to have surface areas as high as 2,000 sq m per g of carbon. It is quite apparent that the higher the surface area of a carbon the higher the carbon's general sorptive powers will be.

Upon microscopic examination of the activated surface, it was found that the surface was quite smooth with a series of sunken pit-like formations covering the entire area. These pits are the entrances to the pores of the carbon.

As well as being a good adsorption media, activated carbon is also known to be an excellent catalyst in many cases. However, it is often difficult to determine, especially during tests, such as were run in this study, whether the actions were physical, or purely chemical with the carbon acting as a catalyst, or possibly a combination of the two. An analysis of the exit gases would possibly have determined whether a chemical reaction was taking place during the process, but this test was not performed since the research was concerned mainly with the physical aspects of the problem.

EXPERIMENTAL PROCEDURE

Dynamic removal was tested in this research rather than static sorption, since it was felt that the more typical uses would be of the dynamic nature rather than of the static type.

Service Time.—In this study, a "Service Time" of the carbon was determined rather than the time that the carbon took to gain equilibrium. Service Time is defined as the time from the moment the gas mixture starts to pass through a carbon bed filter until the first noticeable trace of the gas leaves the filter bed after passing through the bed. On the other hand, the "Dynamic Equilibrium Time" could be defined as that time, from the start of the test, that it takes for the composition of the gas leaving the filter to become the same as the composition of the gas entering the filter. At this point, the carbon has removed all the gas that it is able to remove under the test conditions, and the gas stream is merely flowing over the carbon bed with no action in the carbon at all during this time.

It was felt that the dynamic equilibrium time is not as important in a study such as this, since a large quantity of the gas would have escaped by the time that the system reaches this state of balance. If any hydrogen sulfide escaped, above the detectible limit, the system would not be desirable, because the condition that was to be prevented would then exist. Therefore, the time that the first minute quantity of gas appears on the exit side of the filter is the "life" of the bed. In a practical application, this life would be the factor that would govern the operation of the system.

Instrumentation.—The system used in this research was a completely closed system, as shown in Fig. 1. The hydrogen sulfide that was used in this study was a technical grade obtained from a standard lecture bottle, and was regulated at the bottle by means of a needle valve. Immediately past this point, a two-way valve (No. 2, Fig. 1) was placed in the system. On one leg of this valve a rotometer type instrument, with a range for gases of 3 cu cm per min to 60 cu cm per min, was placed for the purpose of measuring the hydrogen sulfide flow rate. Since the concentrations of the gas were held at relatively low magnitudes, the instrument used covered the desired range quite adequately. This meter is represented as No. 3, Fig. 1.

The meter was calibrated in the laboratory for the tests by the method of air displacement of water. Hydrogen sulfide would have been the ideal gas to use in this displacement, but since hydrogen sulfide either reacts with, or is soluble in, nearly every liquid available, it was not practical to use; therefore, air was the gas used to displace the water and calibrate the meter, since air will not react to an appreciable extent with water. The resulting calibration curve for the meter agreed very closely with that supplied by the manufacturer at the higher flows, but as the flow was decreased, the meter became increasingly inaccurate. Since the meter was guaranteed only to be accurate to within 2% at the maximum scale reading, the inaccuracy was reasonable and accountable.

When the hydrogen sulfide was used in the meter during the actual tests, the rate that was read for the meter was corrected because of the difference in specific weights of air and hydrogen sulfide.

The hydrogen sulfide flow rate was measured with the gas discharging from the meter into air. When the test was started, the two-way valve (No. 2, Fig. 1) was turned so that the gas would then go through the system. Although the system pressure was somewhat less than atmospheric, no appreciable error was found to exist by measuring the hydrogen sulfide flow rate as described previously. In order to verify the accuracy, the pressure in the system was measured and, with this pressure, a correction factor was determined to be insignificant.

During some of the runs, the total amount of gas that was used was quite large, thus reducing the pressure in the hydrogen sulfide lecture bottle. This reduction in pressure caused a slight reduction of the flow rate of the gas. Because of this complication, the flow rate of the hydrogen sulfide was checked at about five minute intervals throughout each run. In order to do this check, the gas had to be turned off the system and allowed to go through the flow-meter and discharge into the air. A slight degree of inaccuracy was caused by not having the gas in the system the complete time, but the reading took only about five seconds at the longest to complete. The resulting error caused by this check reading was about 1%, a value well within the accuracy of the research as a whole.

It should be noted that the decrease in flow rate from the start of a run to the end of the run was quite uniform throughout all the tests.

Air Intake.—The air intake during all but the 100% humidity runs was through a rotometer type flow meter (No. 13, Fig. 1) that registered from 0.5 to 15 l per minute. This meter was equipped with valves on either side to regulate the flow. The fact that this gage is completely enclosed, thus allowing no losses either before or after the ball float, should also be mentioned.

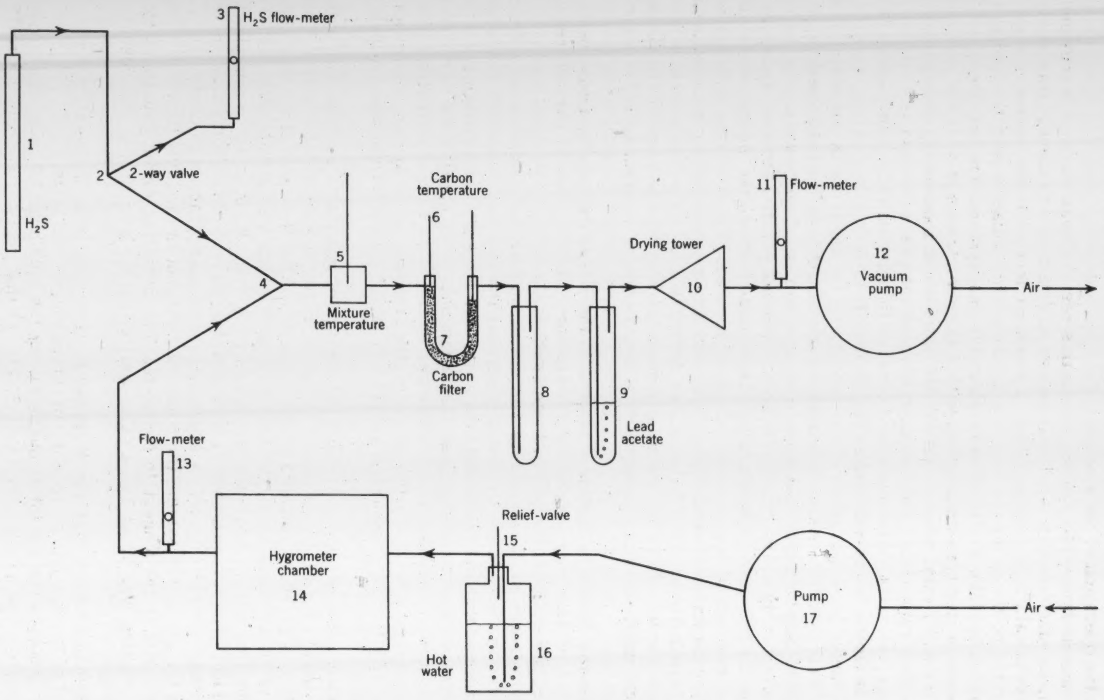
1
H_2S
2
2-way valve
3
H_2S flow-meter
4
5
Mixture temperature
6
Carbon temperature
7
Carbon filter
8
9
Lead acetate
Drying tower
10
11
Flow-meter
12
Vacuum pump
Air
13
Flow-meter
Hygrometer chamber
14
Relief-valve
15
16
Hot water
Pump
17
Air

FIG. 1.—INSTRUMENTATION

Mixing.—The movement of the air and the hydrogen sulfide was effected by the vacuum pump (No. 12, Fig. 1) that pulled the mixture through the system. A wye connection (No. 4, Fig. 1) was installed connecting the hydrogen sulfide line with the air line. This connection served as a mixing point for the air and the hydrogen sulfide gas.

Since the hydrogen sulfide was never greater than 2% of the total mixture, sufficient mixing took place prior to the filter bed as determined by the Reynolds Number. At least 1 ft of tubing separated the wye and the filter, and, the flow being turbulent, good mixing should have taken place.

Mixture Temperature.—No. 5, Fig. 1 shows the relative location of the point at which the mixture temperature was determined. The apparatus used was a flask into which the flow had to pass over the bulb of a thermometer before leaving the flask. In addition to being a temperature device, this flask also acted as a point for additional mixing of the hydrogen sulfide and air.

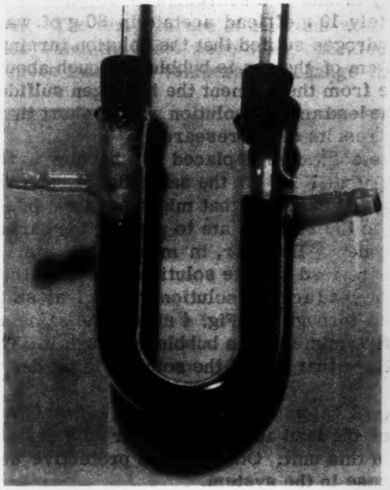

FIG. 2.—U-TUBE WITH CARBON SHOWING THERMOMETERS FOR CARBON TEMPERATURE DETERMINATION

Activated Carbon Filter.—The mixture was then brought into a calcium chloride U-tube in which the sample of activated carbon was placed. The filter is shown as No. 7, Fig. 1. Thermometers were placed in the carbon, both near the entrance and exit of this tube, so the carbon temperature could be determined throughout the run, as shown in Fig. 1, No. 6 and in Fig. 2.

As a means of trying to determine if the shape of the filter bed had any effect upon the sorptive properties of the activated carbon, a series of tests were run using a long straight tube rather than the U-tube. After finding that the filter bed shape made no difference in the results, the U-tubes were used throughout the remainder of the tests, since these tubes were more convenient to use in the determination of the carbon entrance and exit temperatures.

Head Loss.—A differential water manometer was attached across the filter bed to see what effect the removal process might have upon the head loss across the bed. It was not the purpose of this instrument to determine what head losses might be expected if a unit were actually installed, but rather to see if the head loss across the filter changed due to gas removal taking place in the bed. The actual loss measured was somewhat greater than that caused by the carbon alone, since the manometer was tied into the system 3 or 4 in. before and after the filter bed. Since all other factors in the system remained relatively constant, the differential changes were caused by the actions in the carbon alone.

Lead Acetate Indicator.—The hydrogen sulfide was detected on the exit side of the apparatus by means of a concentrated solution of lead acetate. This solution, approximately 10 g of lead acetate in 80 g of water, is so sensitive to the presence of hydrogen sulifed that the solution turning to black is detectable when about 0.1 cu cm of the gas is bubbled through about 3 ml of the solution. The length of time from the moment the hydrogen sulfide left the filter until it was detected by the lead acetate solution was so short that no appreciable error was caused in the results of the research.

The lead acetate solution was placed in a bubbler in the system as shown in Fig. 1, unit No. 9. Unit No. 8, in the same figure, was used as a safety device to catch any of the lead acetate that might escape. In some runs, this device was placed prior to the lead acetate to protect the carbon in case of a breakdown in the equipment. However, in most of the runs, this unit was placed downstream from the lead acetate solution to protect the drying tower.

Fig. 3 shows the lead acetate solution in its clear state before any hydrogen sulfide was passed through it. Fig. 4 shows the same solution 3 sec after an air-hydrogen sulfide mixture was bubbled through it. The amount of hydrogen sulfide in the mixture that turned the solution to the deep black shade was less than $\frac{1}{2}$ cu cm.

Drying Tower.—A calcium chloride drying tower (No. 10, Fig. 1) was placed in the system after the lead acetate indicator as a device to protect the equipment that followed this unit. Other than a protective device, this unit served no functional purpose in the system.

Mixture Flow Meter and Control.—After the calcium chloride drying tower, another Hoke gage (No. 11, Fig. 1) was installed so that there would be a check on the total flow rate through the system. With a gage at the beginning and end of the system, it was possible for any leaks that might develop in the chain to be readily detected, since both these gages should read the same unless some air were entering between them. The hydrogen sulfide, that entered between these gages, was such a small percentage that it was unable to be detected on these gages.

Since all the air and hydrogen sulfide was being pulled through the system by means of the vacuum pump (No. 12, Fig. 1) at the end of the chain, the total flow rate was adjusted by a valve placed between the pump and the rate meter.

High Humidity Device.—During the runs that were made for high humidity determinations, another apparatus was built and attached to the entrance side of the air intake of the first flow gage (No. 13, Fig. 1). The pressure side of another vacuum pump (No. 17, Fig. 1) was used to discharge into a partially filled 5 gal jar that had hot water in it (No. 16, Fig. 1). The passage of the air through this water caused the air to pick up a great deal of moisture as well as to become heated. The exit line of this bottle was then put into a sealed chamber (No. 17, Fig. 1) in which the air was allowed to pass across a wet and dry bulb hygrometer. A relief valve (No. 15, Fig. 1) was put on the bottle to prevent excessive pressure build-up in the system. Fig. 5 shows this protion of the system under operating conditions.

When the air reached a state of 100% humidity the test run was started. Since there were about 3 ft of tubing between the hydrometer chamber and the mixture temperature chamber, the air temperature was lowered from about 7° to 10° by the time it reached the carbon bed. With this lowering of the temperature, and with 100% humitidy condition prior to this, the humidity of the mixture at the point at which it entered the filter bed was probably near to 100%. Because of the lowering of the temperature, it is certain that the humidity was at all times just at 100%, since there was some condensation prior to the filter.

Temperature Control.—The temperature of all runs was controlled by the room temperature under which the tests were being operated. During any one run, there was never more than 1° C temperature change throughout the run.

FIG. 3.—LEAD ACETATE SOLUTION IN CLEAR STATE

FIG. 4.—SAME SOLUTION SHOWN IN FIG. 3 AFTER H_2S CONTACT

During one series of tests, in which high air temperature was needed in large amounts, the air in the room was drawn into the system by sealing the room and heating all of the air contained therein. The temperature of the room was raised high enough so that the air entering the filter was at a temperature of 100° F.

RESULTS

The work done in connection with this research used three different types of carbon. These carbons will be designated as Type I, Type II, and Type III. The majority of the work was done with Type I carbon since this carbon was more readily available. The other carbons were used in order to determine whether their sorptive properties were the same under the same conditions of experimentation.

Properties of the Carbons

Type I.—Type I carbon is an impregnated extruded granular carbon. The total internal surface area is 1,000 - 1,200 sq m per g of carbon as determined

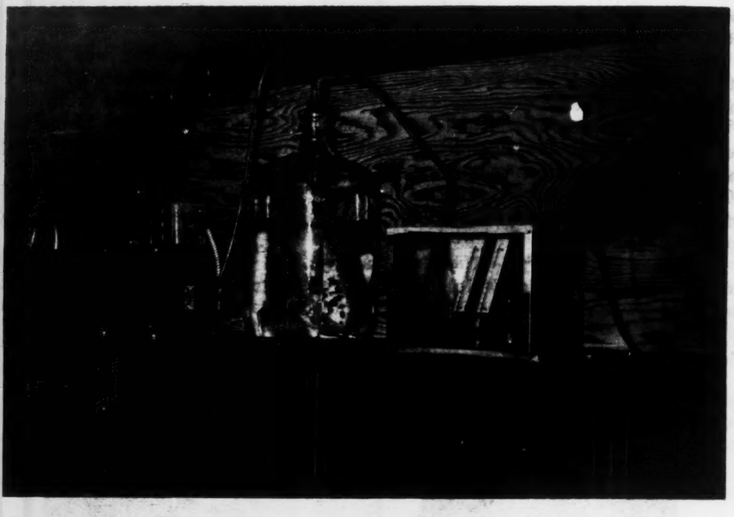

FIG. 5.—HIGH HUMIDITY DEVICE

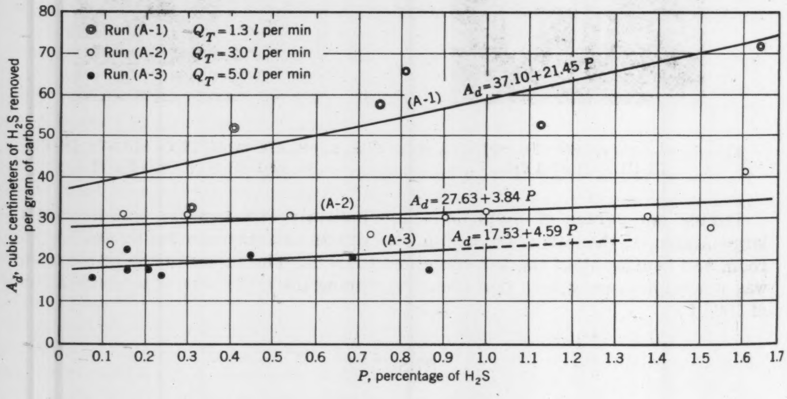

FIG. 6.—H_2S REMOVED VERSUS PERCENTAGE H_2S

by the Brunauer, Emmet, and Teller (BET) method. The ash content is 5% - 10%, water solubles 2% - 5%, pH in water 9±, and the apparent density is between 0.30 and 0.50 g per cu cm. This carbon is impregnated with copper sulfate to a copper content of approximately 4% by weight.

Type II.—Type II carbon is also a copper impregnated carbon. However, the copper content of this carbon is not known, but it was assumed to be about the same as that of the Type I carbon. The total surface area by the BET method if 1,050 - 1,150 sq m per g of carbon, the apparent density is 0.50 g per cu cm, the pore volume is 0.8 cu cm c per g, the specific heat at 100°C is 0.25, the maximum ash content is 0.8%, and the maximum moisture is 2.0%. The size of this carbon is graded, according to the manufacturer, such that it will pass a 12 and be retained on a 30 U. S. Sieve. However, some of the carbon was found to pass a 200 U. S. Sieve indicating that the carbon is not capable of withstanding abrasion or rough handling in shipping.

Type III.—Type III carbon is exactly the same as Type II carbon in all respects including manufacture except that it is not copper impregnated. The tests using this carbon were run to determine whether the copper impregnation was significant in the H_2S removal process.

Effect of Variable Flow.—A group of tests were run to determine what effect a variable total flow would have upon the sorptive properties of Type I carbon. All the tests of this group were performed using the calcium chloride U-tube filter with 10 g of carbon on all runs. The data for this group of tests, Group-A, can be found in Table 3(a). A series of three runs were made with total mixture flow rates (Q_t) of 1.5, 3.0, and 5.0 l per min passing through the filter. These runs are designated as runs A-1, A-2, and A-3 respectively.

Each of these runs are made up of a series of individual tests in which the percentage of hydrogen sulfide in the total flow was varied.

There appears to be some significance between the hydrogen sulfide percentage concentration (P) and the cubic centimeters removed per gram of carbon (A_d) for the results of this group. The least square lines for these quantities were computed in an effort to determine what influence one variable has upon the other. The resulting lines can be seen in Fig. 6. Although there is some doubt as to the actual significance of this line as a predicing device, it is interesting to note that all the lines have slopes that are positive, indicating that as the percentage hydrogen sulfide increases, the carbon will remove more of the gas.

Although the lines intersect the ordinate at a point that is greater than zero, it is apparent that this is impossible, since the carbon can not adsorb any hydrogen sulfide unless it is present in the air stream. Therefore, the least square lines were stopped just short of the ordinate, since there is no evidence as to the shape of the curve between this point and the origin.

The means, standard deviations, and correlations between the percentage of hydrogen sulfide in the total flow (P) and the cubic centimeters of hydrogen sulfide removed per gram of carbon (A_d) are presented in Table 1.

As seen in Table 1, the statistical parameters are in themselves quite irregular. Run A-1 has a large standard deviation when compared to that of the other two runs of this group. However, the correlation coefficient of this run is quite large, denoting that there probably is a significant straight line through the points within the limits of the data. With this correlation coefficient, the large standard deviation seems quite reasonable, since a large standard deviation is an indication that the sample values are not from the same sample population.

The converse of the preceding is true for Runs A-2 and A-3, as can be seen in Table 1. The standard deviations of these runs are quite small, as are their correlation coefficients. Therefore, all the data in these runs could possibly have come from the same population with the variation due to experimental error only.

The increased sorptive properties might also be explained by the heat generated in the system as the hydrogen sulfide concentration became greater. A discussion of this hypothesis can be found elsewhere in this paper under the heading of "Temperature Effects upon Hydrogen Sulfide Removal."

Fig. 7 shows the relationship of the means $\bar{A}_d$ of Runs A-1, A-2, and A-3 plotted against the total mixed flow rate Q_t passing through the filter. From this line, it can be theorized that as the flow rate becomes greater the amount of hydrogen sulfide that is removed will become less. The hypothesis supports the theory as stated under the heading "Adsorption Theory." Also shown on Fig. 7 are other lines from some of the other groups of runs made during this research. These other lines support the same theory as does the line for Group A.

TABLE 1.—STATISTICAL BREAKDOWN FOR GROUP A

Run No. (1)	Mean of cu cm H_2S removed per gram, $\bar{A}_d$ (2)	Standard deviation of cu cm H_2S removed per gram (3)	Correlation coefficient of A_d and P (4)
A-1	55.1	12.5	0.772
A-2	30.8	2.5	0.488
A-3	19.4	3.1	0.411

From the data presented thus far, it is quite apparent that activated carbon might be used as a sorbent medium if the flow rates through the material are held to a relatively low value.

Effect of Variable Filter Column Length.—A series of tests were made to determine what the effect on the sorptive rate per gram would be if the length of the filter were altered. In this series of tests, designated as Group-B, the total mixed flow rate Q_t was held constant throughout all runs, although the hydrogen sulfide percentage varied. The diameter of the filter was also constant, whereas the amount of carbon put into the filter was varied in the test. A series of runs were made with 5.0 g and 15.0 g of activated carbon in the filter at a total flow rate of 3.0 l per min, providing a column length of 5.5 and 18.5 cm respectively. The results of Run No. A-2, Group-A will also be used with the results of Group-B, since this run used the same carbon and the same flow rate. The tabulated results for Group-B can be found in Table 3(b).

A statistical breakdown of the results of this test are shown in Table 2. In this table, the mean hydrogen sulfide removed per gram of carbon $\bar{A}_d$, standard deviation of the hydrogen sulfide removed per gram, and the correlation between the hydrogen sulfide removed per gram A_d and the percentage hydrogen sulfide in the total flow are shown. These same parameters are also included for Run No. A-1 for comparative purposes.

The standard deviations for this group of runs is quite large, when compared to the magnitude of the mean for the same runs. Run B-1 has a large correlation coefficient indicating that this run probably has some significance when related to the percentage hydrogen sulfide in the gas stream passing through

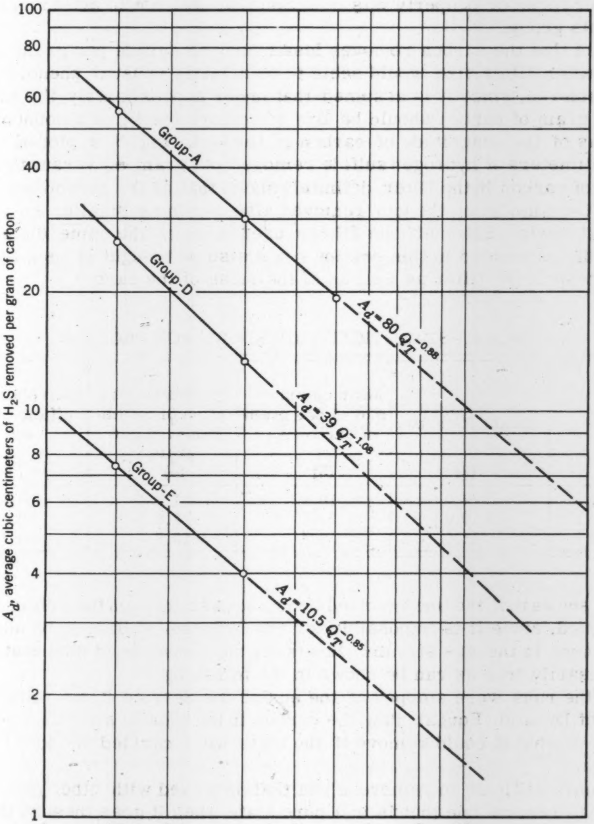

FIG. 7.—AVERAGE H_2S REMOVED VERSUS TOTAL FLOW RATE

the filter. Therefore, the high standard deviation in this run follows the general trend, as pointed out in discussion of the runs in Group-A.

The statistical parameters for Run A-2 have previously been discussed.

The significant factor in Run B-2 is the very low correlation coefficient. Upon studying the data for this run, it can be seen that Run No. B-2, 3 has a very large value for the cubic centimeters of hydrogen sulfide removed per gram of carbon, when compared to these same quantities for the remainder of the group. The cause of this high degree of removal was not able to be determined in the laboratory at the time of the test, and no apparent errors in the instrumentation could be detected at that time. It is, therefore, not known whether this high degree of removal was caused by some error, or whether some other force or property was in action that was not in action on the other runs of this group.

The fact that the carbon removes less hydrogen sulfide per gram of carbon at the smaller filter size would seem to be a fairly unusual phenomena when first considered, since it is assumed that under approximately the same conditions, a gram of carbon should be able to remove the same amount of the gas regardless of the magnitude of carbon in the bed. Fig. 8, a plot of the mean cubic centimeters of hydrogen sulfide removed per gram $\overline{A}_d$ versus the number of grams of carbon in the filter, definitely shows that as the carbon bed becomes smaller, the amount of the gas removed also becomes smaller on a straight line relationship. Since all the filters used were of the same diameter, the relationship presented in this section could also be thought of as a function of the thickness of the filter as well as of the mass of the carbon.

TABLE 2.—STATISTICAL BREAKDOWN FOR GROUP-B

Run No. (1)	Gram of carbon (2)	Mean cu cm removed per gram of carbon $\overline{A}_d$ (3)	Standard deviation of cu cm removed per gram (4)	Correlation coefficient of A_d and P (5)
B-1	5.0	10.1	7.2	0.81
A-2	10.0	30.8	2.8	0.49
B-2	15.0	50.4	13.8	0.23

Fig. 8 shows that the line involved does not pass through the origin, as might be presumed, since it is impossible for any hydrogen sulfide to be adsorbed if there is none in the gas stream. However, the converse of this statement is not necessarily true as can be shown in the following.

Since the runs were stopped at the end of the Service Time rather than at the time of Dynamic Equilibrium, the carbon in the beds in some cases removed only part of what it could remove if the tests were carried out to the equilibrium time.

It is more difficult to remove a gas that is mixed with other gases than it would be to remove one that is in a pure state when it goes through the filter. Since this testing was performed in an air-hydrogen sulfide mixture, the removal was hampered to some degree. This retarding factor was also pointed out in the tests that had a variable flow rate. It is, therefore, hypothesized that a family of curves could be developed similar to that shown in Fig. 8 for a series of different flow rates. For a total flow rate greater than 3.0 l per min, the curves would be parallel to the one shown, but would intercept the abscissa at a point greater than the point at which the 3.0 l per min flow rate intercepts it. Also, if the flow rate were less, the line should intercept the abscissa at a point nearer to zero.

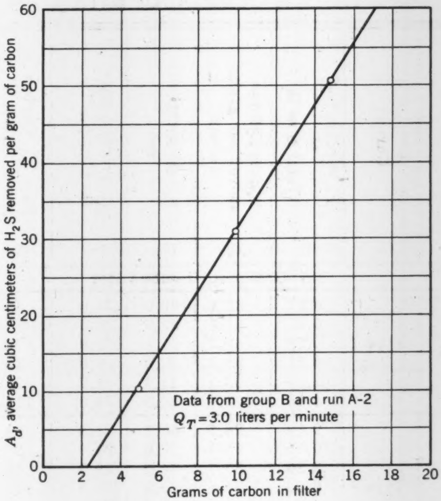

FIG. 8.—AVERAGE H_2S REMOVED VERSUS CARBON FILTER SIZE

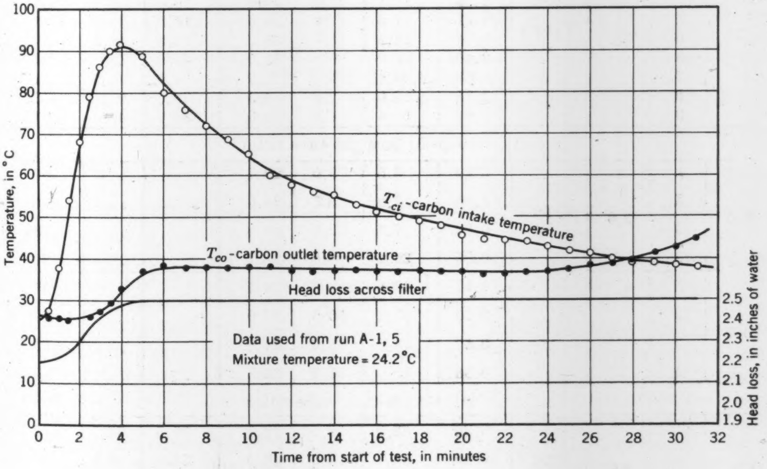

FIG. 9.—CARBON TEMPERATURES AND FILTER HEAD LOSS VERSUS TIME

TABLE 3.—SUMMARY OF DATA

Test (1)	Number (2)	Q_t, in liters per minute (3)	Carbon (4)	P, Percent H_2S in Total flow Q_t (5)	A_d, cc H_2S removed per gram carbon (6)	T, Service time, min. (7)	t_A, Average Mixture temp., °C. (8)	Max. t_{ci}, Carbon Intake Temp., °C. (9)	Max. t_{co}, Carbon Outlet temp. °C. (10)	H, Relative Atmospheric Humidity, % (11)
(a) Group-A, U-tube filter										
A-1	1	1.5	Type I 10g	0.31	32.1	69.0	24.0	43.0	28.5	40
	2			0.41	51.5	83.0	26.0	- -	34.0	32
	3			0.75	57.3	51.0	25.9	70.0	46.0	35
	4			0.81	65.3	54.0	26.0	68.0	41.5	38
	5			1.13	52.9	31.0	24.2	91.5	45.0	33
	6			1.65	71.6	29.0	24.0	119.0	48.0	30
A-2	1	3.0	Type I 10g	0.12	23.7	66.5	21.3	31.0	29.0	55
	2			0.15	31.4	70.0	24.0	37.0	32.0	43
	3			0.30	30.7	34.0	25.0	40.1	42.9	33
	4			0.54	30.8	19.0	23.0	42.0	53.0	37
	5			0.91	30.0	11.0	23.0	87.0	87.0	36
	6			1.38	31.0	7.5	25.5	108.0	68.0	52
	7			1.53	27.5	6.0	20.5	121.0	58.0	62
	8			1.61	41.1	8.5	21.0	113.0	120.0	39
A-3	1	5.0	Type I 10g	0.08	15.9	41.0	24.5	32.0	30.0	54
	2			0.16	22.6	28.0	22.5	33.0	34.0	56
	3			0.16	17.6	22.0	24.0	36.0	35.7	54
	4			0.21	17.5	17.0	24.0	35.6	38.0	58
	5			0.24	15.9	13.0	23.7	37.0	45.0	47
	6			0.45	21.2	9.5	23.0	49.5	58.0	44
	7			0.69	20.6	6.1	25.0	71.0	78.0	51
	8			0.73	25.6	7.0	24.5	55.0	72.0	73
	9			0.87	17.4	4.0	24.6	85.0	48.0	51
(b) Group-B, Straight-tube filter										
B-1	1	3.0	Type I 5g	0.19	3.5	3.0	23.4	. . .	. . .	69
	2			0.36	11.8	5.5	24.6	. . .	. . .	36
	3			0.60	7.2	2.0	24.9	. . .	. . .	43
	4			0.85	20.2	4.0	24.7	. . .	. . .	37
B-2	1	3.0	Type I 15g	0.17	47.8	138.0	24.9	. . .	. . .	37
	2			0.26	47.1	90.0	23.9	. . .	. . .	39
	3			0.57	74.0	65.0	24.0	. . .	. . .	33
	4			0.96	49.7	26.0	23.5	. . .	. . .	42
(c) Group-C, U-tube filter										
C-1	1	3.0	Type I 10g	0.35	42.9	41.0	38.8	56.5	52.0	38
	2			0.45	44.7	33.0	40.5	63.0	56.5	43
	3			0.80	33.4	14.0	37.5	67.0	64.5	36
	4			0.90	43.3	16.0	37.2	76.6	69.0	40

TABLE 3.—CONT'D

Test (1)	Number (2)	Q_t, in liters per minute (3)	Carbon (4)	P, Percent H_2S in Total flow Q_t (5)	A_d, cc H_2S removed per gram carbon (6)	T, Service time, min. (7)	t_A, Average Mixture temp., °C. (8)	Max. t_{ci}, Carbon Intake Temp., °C. (9)	Max. t_{co}, Carbon Outlet temp. °C. (10)	H, Relative Atmospheric Humidity, % (11)
(d) Group-D, U-tube filter										
D-1	1	1.5	Type II 10g	0.74	23.3	21.0	26.7	40.0	37.5	49
	2			1.31	27.1	13.8	22.4	44.3	46.0	53
	3			2.24	73.7	22.0	24.0	51.0	145.0	50
D-2	1	3.0	Type II 10g	0.23	15.2	21.5	23.3	27.1	34.4	50
	2			0.30	12.7	14.0	23.8	29.9	39.0	50
	3			0.41	9.0	7.4	23.8	29.5	37.0	50
	4			0.61	10.9	6.0	22.8	30.0	40.0	49
	5			0.72	11.9	5.5	22.2	32.0	40.0	49
	6			1.56	10.2	2.2	24.0	34.7	48.0	50
(e) Group-E, U-tube filter										
E-1	1	1.5	Type III 10g	0.71	7.2	6.8	25.1	36.0	41.0	- - -
	2			1.30	7.8	4.0	25.1	37.5	50.0	55
E-2	1	3.0	Type III 10g	0.22	3.9	6.0	24.4	26.1	32.0	48
	2			1.19	4.5	2.5	27.1	30.1	38.0	47
(f) Group F, U-tube filter										
F-1	1	3.0	Type II 10g	0.32	52.3	55.0	25.6	45.0	40.0	100
	2			0.67	20.0	10.0	26.6	49.4	45.6	100
	3			0.74	21.2	9.5	26.0	49.6	53.0	100
	4			1.10	24.9	6.5	26.0	55.0	55.0	100

The magnitude of carbon at the point of intersection of the line on Fig. 8 is also thought to be of significance. In the filter used in this test, the minimum quantity of carbon at which removal will take place without a zero service time is 2.4 g. If any amount less than this were to be put into a filter, the service time would be zero, as some of the hydrogen sulfide would not be removed and would come through the filter. The distance that this magnitude of carbon occupies can, therefore, be thought of as the length that is needed under the conditions of this test to completely remove the residual hydrogen sulfide that does not become removed in the carbon previous to that layer. A more complete discussion of the residual gas and the mass of carbon needed to gain proper removal will be made in the next section that deals with the temperature changes taking place as the hydrogen sulfate is removed. It will be shown that the temperature increases are a direct function of the hydrogen sulfide removed, and that the temperature changes take place in a regular manner.

It must be remembered that the tests discussed in this section are for one total flow rate, one filter size (diameter), and one type of carbon. Other filter sizes, carbons, and flow rates should be tested to verify some of the "family" relationships found during this research, as it is possible that other parameters might be found that will increase the knowledge of the properties needed for a practical installation.

Temperature Effects Upon Hydrogen Sulfide Removal.—At the very start of the experimentation, it was noticed that the carbon became warm, and in some cases hot, during the sorption process. Because of this occurrence, thermometers were placed in the carbon both at the entrance and exit of the U-tube filter. Although the temperature recordings that were obtained from these thermometers can not be compared precisely from one run to another, because the bulbs of the thermometers were not set in the carbon the same distance on each run, the data obtained serves the purpose of presenting another hypothesis concerning the sorptive power of activated carbon.

At the start of a run, the temperature of the carbon will immediately start to rise in the carbon at the entrance side of the filter. As the run progressed, this temperature, designated t_{ci}, would reach a maximum and then slowly recede throughout the rest of the run. The percentage hydrogen sulfide in the air stream was also found to be a function of how great t_{ci} would become. On some runs, the carbon would become so hot that it would start to glow very brightly. This temperature was beyond the range of the thermometers used, therefore, no data could be obtained as to how great the heat actually was. This increased heat phenomena was not limited to just one test, but occurred on all runs of the experimentation in varying magnitude.

It should be pointed out that the heat never became as great on the runs having a total flow rate of 5.0 l per min as it did on those runs whose flow rate was less than this value. The reason for this occurrence is that the higher flow rate tends to act as an air cooler and does not allow the heat to build up to such a great extent by dissipating it through the bed.

The hot layer of carbon that started at the entrance to the bed traveled around the U-tube toward the exit as the test progressed. The exit thermometer temperature, too, was seen to rise quite soon after the start of the test and then remain relatively constant for most of the remainder of the run. The initial rise of the exit thermometer was undoubtedly caused by the air being heated at the start of the filter and remaining hot until after it had passed through the filter, thus providing the heat that caused the temperature to rise at the exit.

Toward the end of a run, the exit temperature would start to rise in a manner similar to that which occurred at the entrance to the carbon at the start of the run. In most cases, it was at this point that the first traces of the hydrogen sulfide started to turn the lead acetate indicator dark; this darkening indicates the srevice time and the end of the test. However, during some runs, the exit thermometer was set deeper into the carbon, and the exit temperature would then reach the same value as the entrance temperature had at the start of the run. A typical case of the temperature rises is shown in Fig. 9 for one run.

Also shown on Fig. 9 is the head loss across the filter plotted against time from the start of the test. The head loss was determined merely as a function of the time and not as an indication of how great the loss would become for any particular carbon. It was seen that the head loss rose until the time at which the maximum temperature was attained in the entrance section of the filter, at which time the loss remained constant throughout the remainder of the run.

The apparent significance of this loss is evidenced by the fact that it remained constant after the first part of the run started. The theory to explain this constant loss is that the carbon expands when it becomes heated during the sorption process, and, therefore, with the total flow rate remaining constant, the head loss is increased. When the maximum temperature is attained in the filter, the maximum head loss occurs at approximately the same time. After the temperature starts to subside, the carbon then shrinks back to its original size at the first section, while it expands at the next section in the filter.

As was pointed out in the preceding section, the temperature starts to drop immediately after attaining its maximum on the entrance side of the filter. It may be theorized that when the temperature drops in a section of the carbon all removal possible in that section has taken place, and the next section will begin its total removal process. Therefore, the process might be considered to consist of a band traveling around the filter bed, starting at the entrance and ending at the exit side of the filter at the end of the run. It was pointed out in regard to Fig. 8 that there must be a certain amount of carbon in the bed before any removal would take place. This section, or length or carbon, will be called the residual section, as it is the section that can not obtain total equilibrium with respect to the amount of hydrogen sulfide entering and leaving the section without the Service Time being zero.

Fig. 10 represents the hypothesis that is thought by the authors to be taking place in the sorption process. An air-hydrogen sulfide mixture enters the filter, and the carbon begins to rise somewhat in temperature. After the carbon has reached its maximum temperature at the entrance, a band of high temperature will begin to progress through the filter, with each increment of length presumably reaching this same maximum temperature. As the band moves, the carbon in the band will remove all the hydrogen sulfide it is capable of removing under those set conditions and leave behind it the carbon that is in equilibrium with respect to the incoming and outgoing gas in that particular section. Immediately following this high temperature band is a band that removes nearly all the residual hydrogen sulfide that was not removed in the high temperature band. This residual section is thought to be of the same size as the section determined by the intercept on the abscissa of Fig. 8. Following the residual section is that section of carbon that has only air passing through it at any instant.

Therefore, the filter bed is made up of a section that is in equilibrium with the hydrogen sulfide, an active removal section, a residual hydrogen sulfide removing section, and a section having only air passing through it. Once the moving band reaches the point at which the leading boundary of the residual section is at the exit of the filter, the hydrogen sulfide will appear in the exit stream, and the Service Time will have been reached, since a small quantity of hydrogen sulfide will escape at this point and be detected.

The high temperatures in the carbon bed are thought to be caused by the oxidation of the hydrogen sulfide forming another sulfur compound on the carbon itself. Undoubtedly, the largest amount of the hydrogen sulfide gas is removed from the flow by this process that would be termed chemisorption rather than adsorption. However, some of the gas is undoubtedly adsorbed in the strict sense of the word. Therefore, more than one process is thought to be taking place in the filter at all times.

A microscopic examination was made of some of the carbon that was "burned" on a run when the carbon glowed. It was noticed that, on the visible portions of this carbon, some very small particulate material was present. Upon comparing this to carbon that had not been used in the process, it was noticed that these particles were not present on the unused material. The conclusion based upon the preceding facts is that the particles must be a chemical compound formed in the removal process.

There is a possibility that a strict chemical reaction is also taking place as well as the sorption processes previously mentioned. Since equipment was not available for the exact determination of what the actual cause of the hydrogen sulfide removal actions were, little is able to be said concerning these properties that are chemical in nature.

During any set of runs, it was noticed that the greatest removal took place during the test that had the largest percentage of hydrogen sulfide in the total flow. It was also noticed that, in most cases, the highest carbon temperatures also occurred during these same runs. Because of this occurrence, it may be presumed that a series of runs at high temperatures, using the same carbon as previously used, would prove beneficial to the understanding of the action in filter.

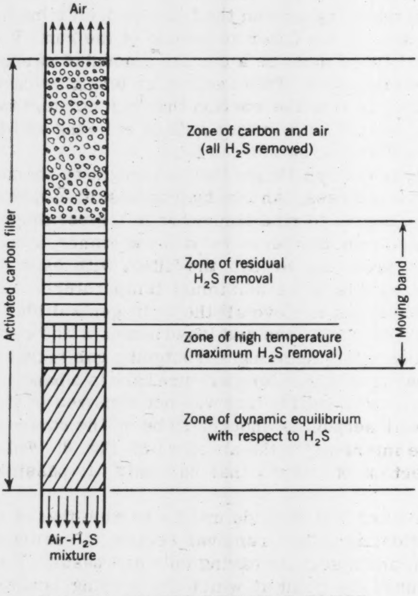
Air
Activated carbon filter
Zone of carbon and air
(all H_2S removed)
Zone of residual
H_2S removal
Moving band
Zone of high temperature
(maximum H_2S removal)
Zone of dynamic equilibrium
with respect to H_2S
Air-H_2S
mixture

FIG. 10.—FILTER REMOVAL ZONES

The room in which the tests were being made was warmed to a temperature of near 100° F, for one group of tests to simulate a relatively high temperature condition that might be found in the summer months in some parts of this country. The air-hydrogen sulfide mixture was, therefore, at this temperature upon entering the carbon bed during any one run.

The results of this series of tests, Group-C, can be found in Table 3(c). Since the runs used 10.0 g of Type I carbon at a total flow rate of 3.0 liters per minute, the results of this group will be compared to the results of Run No.

A-2, that was run under the same conditions, except that the temperatures of the mixture were somewhat lower.

The mean value of the cubic centimeters removed per gram of carbon in Group C-1 was found to be 41.1. The corresponding value for Run A-2 was 30.8. Therefore, the runs at an elevated temperature removed approximately 33% more hydrogen sulfide per gram of activated carbon.

The means of the mixture temperatures in runs A-2 and C-1 were 22.0° C and 38.5° C respectively, a 68% increase in temperature for run C-1.

Since the temperature of the incoming mixed flow to the filter proved to be an important factor in the amount of hydrogen sulfide that may be removed by this specific carbon, a great number of applications of a filter bed of this type might be in processes operating at an elevated temperature.

The most logical reason for the increased temperature causing a greater degree of removal could be explained by the fact that the process is really a chemisorption process to a large extent, and that heat tends to increase the chemical reactions that are taking place. An increase in heat decreases the adsorptive powers of a carbon, but aids in chemical reactions, thus supporting the hypothesis that the process is one of chemical reaction.

Further study on this phase of the process is definitely needed. In order to gain a better understanding of the temperature relationships in the filter, a series of recording themocouples should be placed in the carbon bed itself so that a definite pattern could be developed for a series of different conditions and different carbons.

Another important phase of the process that should be investigated is that all conditions of the runs be held constant except for the mixture temperature. The mixture should be controlled from a low of near 32°F to a high temperature of near 200° F. With a series of runs using this information, a defitite relationship could be established for the temperature factors involved in the removal process.

Investigation of Type II Carbon.—Since Type II carbon is quite similar to Type I carbon in many respects, except that it was supplied by a different manufacturer, a series of tests was run to see if there would be an appreciable difference between the two types. The standard U-tube filter was used on these runs, as well as the same amount of carbon on all runs, namely 10.0 g. Two separate runs were made under different total flow rates. These flow rates were 3.0 and 1.5 l per min total flow. The results of this series of tests can be seen in Table 3(d). The group has been designated as Group-D.

The third run of D-1 will not be considered in the following discussion, since it has a removal value that is a great deal larger than any of the other runs in this group. A possible reason that this run was so high might be a breakdown of the equipment such as a clogged line leading into the system from the hydrogen sulfide lecture bottle or some other breakdown in the equipment.

The mean values, omitting Run D-1, 2, are 25.2 and 11.6 cu cm of hydrogen sulfide removed per gram of carbon for Runs D-1 and D-2, respectively. The means for this test are significantly lower than the means found for Runs A-1 and A-2, as reported previously. The general trend of this difference can be seen in Fig. 7, because of the smaller value of the intercept.

Extensive tests were not conducted beyond this point, since the reason for making this series of runs was to determine whether there was or was not a difference in the two carbons. Since it is felt that a definite difference has been shown, the tests were not carried out further.

The significance of the results of this series of tests is to show that, in all probability, all the carbons that are to be considered for use should first be tested to see whether they fall into the general removal level desired.

This same difference in removal properties may also be true for the same carbon obtained from the manufacturer at different times. Although no specific series of tests were made, a sample of Type I carbon was obtained from the manufacturer at a time difference of about six months from the time the first sample was obtained, and it appeared to have much less removal power than the original sample. The indication here is that close quality control has probably not been observed in the manufacture of the carbon.

Investigation of Type III Carbon.—Since both Types I and II carbons were impregnated with copper, it was felt that a carbon should be tested that was not so impregnated in order to determine whether the copper was aiding the removal. Group E, the results of which are in Table 3(e), is an attempt to determine the extent to which the copper helps the removal process.

Type II and Type III carbons are exactly the same in all respects, except that the latter is not impregnated with copper. Therefore, the results of this Group will be compared with Group D. All conditions were held as close to the same in Group E and Group D, so that a proper comparison could be made. However, the authors have no knowledge of whether or not the material was made from the same batch of raw material.

The mean cubic centimeters removed per gram of carbon for Run E-1 was 7.5 and for Run No. A-2 was 4.2. These values are considerably below those determined for the runs in Group D. Again, the reader must be referred to Fig. 7, to see these differences graphically.

For the runs at a total flow rate of 1.5 l per min, Run D-1 had 75% greater removal than Run E-1, whereas at a flow rate of 3.0 l per min D-2 had 64% greater removal than Run E-2. Therefore, copper impregnation may be considered to be a vital factor in the removal of hydrogen sulfide from a hydrogen sulfide - air mixture. The reason may be attributed to the fact that the hydrogen sulfide chemically reacts with the copper, or the copper acts as a catalyst, thus causing a chemical reaction in addition to the removal that is carried on in the process without the copper.

Effect of High Humidity.—One of the more important uses for the removal of small quantities of hydrogen sulfide would be from industrial plants discharging into a humid atmosphere such as may be found in the chemical industry or in large sanitary sewers. The series of tests run to determine what effect humidities near 100% would have upon the removal properties used Type II carbon, therefore, this group of runs will be compared with Run D-2.

Group F series of runs were run using only one total mixed flow rate of 3.0 l per min and ten grams of carbon in the filter. The results can be seen in Table 3(f).

The cubic centimeters removed per gram of carbon for Group-F was 29.6. There is one test in this group that seems to be somewhat high, this being Run F-1. If this run were omitted from the computations, the mean removal of cubic cemtimeters of hydrogen sulfide per gram of carbon would be 22.0.

Comparing the preceding figures with those determined in Run D-2, it can be seen that the high humidity created a 50% increase in hydrogen sulfide removal when using the mean value of 29.6. If Run F-1 were omitted, the high humidity would still produce a 47% greater removal rate than the lower humidities.

The apparent increase in removal in this group can not be solely attributed to sorption, because of the fact that during these runs the hydrogen sulfide is probably reacting with the moisture in the air to form a weak sulfuric acid, that, being in a vapor state, would be quite readily removed from the flow through the filter.

It is, therefore, apparent that more hydrogen sulfide may be removed from the stream with the high humidities, but it can not be said that the hydrogen sulfide is being adsorbed by the activated carbon, as the case is probably that the sulfuric acid is being adsorbed, or absorbed.

The percentage of relative humidity at which an increase in removal power takes place was not determined, although it is felt that this is an important factor in the process.

CONCLUSIONS

Activated carbon appears to be of some practical use in the removal of hydrogen sulfide from an air-hydrogen sulfide mixture. However, the process studied in this research indicates a number of limitations as follows:

1. Only small percentages of hydrogen sulfide can be in the air-hydrogen sulfide mixture, or the possibility of fire or explosion could develop due to the heat liberated by the activated carbon during the removal process.
2. For economic reasons, the total mixed flow rate through the filter must be kept to a minimum practical value in order to make the zone of residual removal as short as possible.
3. Due to the fact that nearly all carbons are different when supplied by different manufacturers, each individual carbon should be tested to determine its removal powers and to establish the limitations of its use.
4. The size of the filter structure might have to be of an excessive size, since the contact time is a proportional function to the hydrogen sulfide removal.

The most significant factor determined in the research was that the process was not one of adsorption, but chemisorption, since it was definitely shown that chemical reactions were taking place. This conclusion can be supported by the following:

a. The heat present in all runs of the experiment is typical of chemical reactions.

b. An increase in the heat of the mixture caused an increase in removal, contrary to adsorption theory.

c. Microscopic examination of carbon used in the process revealed solid particles on the carbon surface indicating a chemical reaction.

d. An increase in the moisture present in the air caused an increase in the removal of hydrogen sulfide.

e. As the total mixed flow rate was increased, the carbon removed less hydrogen sulfide, thus revealing the incremental time element typical of chemical reactions.

f. As the carbon bed was reduced in size, holding everything else constant, the carbon removed less hydrogen sulfide per gram.

APPENDIX I.—NOTATION

The following symbols are adopted for use in the paper and for the guidance of discussers.

A_d = Cubic centimeters hydrogen sulfide removed per gram of carbon;

$\overline{A}_d$ = mean of A_d;

H = relative humidity, in percentage;

P = percentage hydrogen sulfide in total flow, by volume;

Q_t = total flow rate of air-hydrogen sulfide mixture, liters per minute;

T = service time, in minutes;

t_A = average mixture flow temperature, in °C;

t_{ci} = carbon temperature at filter intake, in °C; and

t_{co} = carbon temperature at filter outlet, in °C.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3216

DRAINAGE, A VITAL NEED IN IRRIGATED HUMID AREAS

By Albert L. King[1]

SYNOPSIS

Total rainfall in humid areas appears to be adequate for almost all purposes. The rainfall pattern, however, is constantly varying and unpredictable. The variability in amount, rate of fall, and time of rains causes droughts on the one hand and flooding of level lands on the other. Supplemental irrigation solves the problem created by droughts, while adequate drainage systems offer a solution to the drainage problem. Where supplemental irrigation is practiced, drainage is as important a factor as the provision of water for crops.

SETTLEMENT OF HUMID AREAS AND RESULTANT PROBLEMS

Settlement of new countries normally takes place in coastal areas and spreads inland along waterways for several reasons. Rivers which overflow their banks build up the well-known alluvial flood plain. In settling a country, some people move into the flood plain because they associate this feature with geologic ages. The valley is very fertile and is accessible through the river channel when other means of communication or transportation may be lacking, while at the same time it provides one easily defended side of the homestead or settlement. The relative infrequency of devastating floods and the natural optimism of settlers make the occupation of the flood plain a compelling inducement.

Flood plains are more or less level, and they offer the most fertile areas for agricultural pursuits. Many areas are swampy or at least poorly drained;

Note.—Published essentially as printed here, in December, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2676, Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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all are subject to occasional overflow and need protection from floods. The need leads to the construction of levees along major portions of many rivers and also along many of the tributary streams. This work is so extensive that it is a regional problem, rather than local. The lower Mississippi River levees are an excellent example of such a completed project.

These levees, along with associated channel changes, permit flood waters to be carried off without damage to protected areas. They lead, however, to the necessity for somewhat similar protection along tributary streams, and to the provision of improved or additional new drainage facilities in many of these areas. In addition, it is found to be economically feasible in many cases to drain many swampy or lowland portions of the protected areas. The facilities are needed to lower the water level below the normal root zone of crops in the area as well as to carry off excess surface water resulting from heavy local rains. In many fairly level areas, drainage is necessary because: (1) surface water from nearby higher land may collect there; (2) soil structure may be such as to permit underground water to drain from higher to lower land and tend to keep it wet; (3) coastal areas may be subject to the effect of tides; and (4) level land may have thick beds of heavy clay near the surface, especially those lands where much irrigation is applied (as in rice culture).

A good example of an area formerly frequently flooded, and for the most part poorly drained, is the St. Francis River basin in Arkansas and Missouri The basin is immediately west of the Mississippi River and extends from about 50 mi south of St. Louis to about 50 mi southwest of Memphis. It is about 7,000 sq mi in area, with about 1,500 sq mi of the upper basin being hilly. Runoff there is rapid and outflow is controlled to some extent by operations at Wappapello Dam.

The remainder of the basin is more or less level. It was swampy in many places and on the whole was poorly drained. Two factors brought about this condition: (1) in the last several hundred years, the Mississippi River has moved eastward to its present location, leaving a poorly-drained area roughly 30 mi wide and 150 mi long with many horseshoe lakes, sloughs, and swamps; (2) the New Madrid earthquake, which occurred about 150 yr ago, caused much of the land in the upper portion of the area to sink several feet, thus aggravating the drainage problem. The entire basin is protected from Mississippi River floods by a massive levee, but the lower end of the basin is subject to backwater when the Mississippi is at high stages.

The land was so fertile, with markets close by, that action was taken several decades ago to drain the Missouri portion of this very level area. It is covered by a network of drainage ditches, diversion channels, local levees, and other controls, and is now an intensively cultivated agricultural area.

The Arkansas portion of the basin also has many drainage facilities, including an inverted siphon, but it is subject at times to Mississippi River backwater. Plans call for the present St. Francis channel to be used as a "sleeve" through which water from the upper St. Francis will be fed directly into the Mississippi. The remainder of the area, the "interior" portion, will be drained by present tributaries, supplemented by some new channels now under construction. After completion of the project, runoff will flow naturally into the Mississippi, except in high water periods, when it will be pumped over a structure which will block the Mississippi River backwater from the area. The total area in the St. Francis basin thus protected by levees and drained in the manner just described is on the order of two and one-half to three million acres.

Thus, upon the completion of protective and drainage facilities along a stream and its major tributaries, the regional problem, it is necessary for interests in the subtributaries to take similar remedial action, the local problem. This local problem soon reaches out to the individual farm or densely populated area, where coordinated action by all interests is required. This action points directly to the setting up of drainage districts with necessary legal status. Cities in the area are now able to provide adequate means for the disposal of sewage and industrial waste. Farmers can now install drains which meet their individual requirements and be fairly certain that crops will not be flooded or drowned out by a too-high water table. Intensive farming becomes commonplace rather than the exception.

Looking at the overall problem, one is reminded of the human body, an example of possibly the best integrated entity. The main river is the torso, the tributaries are the arms and legs, and the subtributaries and sub-subtributaries are the hands and fingers and the feet and toes.

INTENSIVE FARMING FOCUSES ATTENTION ON WEATHER VAGARIES

Now that his most fertile land is protected from flooding and is well-drained, the landowner begins intensive farming. He plans his program on the assumption that there is enough rainfall in humid areas to produce any desired crop. In a short time, however, the landowner finds that crop yields are quite variable and depend to a large extent upon the pattern of rainfall during the growing season. Although annual rainfall, and in most cases monthly, appears to be more than enough for the growth of crops, he finds that it quite frequently does not fall at the time most needed, and that it often falls at a rate which is not the most advantageous. Droughts occur in most seasons while in those same seasons there are often such heavy or prolonged rains that crops are damaged or destroyed. The drought periods can be overcome by supplemental irrigation, while the runoff from excessive rains can be carried off by a drainage system such as described above.

It is important that every farmer provide adequate drains for each of his fields; it is imperative if he irrigates. With irrigation, soil moisture is maintained at or near the optimum level for best crop growth, and it is necessary that provision be made to take care of any heavy rain which might fall soon after a crop has been irrigated. Otherwise the crop may be destroyed or at least damaged. In addition, adequate drainage will remove soluble salts, prevent stagnation of soil water, make the soil more firm or solid, permit a better supply of air to reach plant roots, and allow the soil to warm up more quickly in spring. A wet soil tends to make roots develop too near the surface; a soil with proper drainage will tend to cause roots to go deeper to a depth where a more constant supply of moisture is available, thus minimizing the effect of rapid moisture changes near the surface.

A complete knowledge of the history of water and its actions will enable the engineer to make the best decisions in irrigation and drainage problems. The nature, behavior, and conservation of water in agriculture are subjects covered in considerable detail in "Water," the 1955 Yearbook of Agriculture, and it would be well for the drainage engineer to become familiar with the many facts presented in the book.

Some factors which determine the crops to be grown in a specific area are the type of soil and its depth, the normal rainfall, the relative humidity, the average and range in temperatures, the length of growing season, the normal percentage of possible sunshine, and the normal wind. Man can do little, or nothing, about these factors except for the fact that he can overcome most of the effects of the variability of rainfall.

A study of the various aspects of precipitation will show facts which the farmer must face to provide optimum moisture conditions in his fields. The study will show (1) that he will experience droughts and will need supplemental irrigation, and (2) that his land will require adequate drainage to take care of heavy rains which fall at more or less frequent intervals.

Although some long-term values must be considered, the analysis is based primarily upon rainfall in the 10-yr period 1949-1958[2] at eight locations, each of which may be considered to be representative of a considerable portion of the humid area which leans toward agricultural pursuits. The locations are Memphis, representative of western Tennessee; Ft. Smith, of western Arkansas; Alexandria, of central Louisiana; Selma, of south-central Alabama; Columbia, of central South Carolina; Albany, of southwest Georgia; and Moline and Mt. Carmel, of northwestern and southeastern Illinois, respectively. The record at each of these stations is an example of the specific conditions an individual farmer might experience. The averages and extremes are applicable to an average farmer in the humid area and will enable him to calculate the advantage or disadvantage of modifying his farming practice. For the greatest benefit, the individual farmer should keep his own daily record of temperature and rainfall. He is familiar with his soil, knows the crops he will grow, and will be able to adjust his schedule to meet weather conditions as they occur.

For this paper, conditions which were experienced at particular locations in the humid area were studied. Conclusions were then drawn after considering averages at eight such points. Much research has been done by others considering averages from a large number of stations in a large area, with valid general conclusions being reached by considering these averages.

DROUGHTS

There is little or no correlation between monthly or annual rainfall and droughts, except when rainfall for considerable periods is much below normal. Serious droughts have been noted in periods when total rainfall was above normal. In many cases, the rain fell at excessive rates, much of it ran off, and crops received little permanent benefit. Daily precipitation values must be studied in detail to evaluate the effect of rainfall.

The moisture requirement of a particular crop is the determining factor in defining a drought period. The minimum number of consecutive days considered a drought will vary widely and will depend upon the particular crop and its stage of development. Cotton is grown in much of the humid area, and for this study, a definition of a drought for cotton is given as an example. The National Cotton Council of America agrees that it is a practical and workable definition. Corn has an earlier and shorter period of maximum growth, and the harvesting period is not as critical as that of cotton. A drought period, for this study, is defined as 14 or more consecutive days with less than 0.25 in. of rain on any one day.[3,4] The number of drought days in a month or year is the sum of the drought days in the various periods in those times. In some of the longer drought periods, some rains slightly heavier than 0.25 in. have been disregarded, as a rain of that amount is totally ineffective in breaking a serious drought. The actual growing season for cotton extends roughly from April through September, with the harvest season extending to about the middle of November.

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[2] "Climatological Data," U. S. Weather Bur., Dept. of Interior, Washington, D. C., for various states and years.

Droughts come in all seasons of the year, although in no regular or predictable pattern. Table 1 shows that a total of 499 drought periods occurred in the 10-yr period 1949-1958 at the eight stations selected for study, with an average of 6.24 periods per station per year. The length of the periods varies from 14 days (the minimum by definition) to 81 days, with an average length of about 23 days per period. The median period is about 20 days. The study shows that location in the humid area appears to have little effect upon the number or length of drought periods. The frequency of drought periods of various lengths is indicated by the mean curve in Fig. 1.

At first glance, it would seem to be sufficient to consider only those droughts which occur during the growing season of the particular crop under study. Further study, however, shows that all droughts during the entire year must be considered for two reasons: (1) a prolonged drought in the spring will reduce the moisture content to such an extent that the soil may be difficult to work. If a seed-bed has been prepared, there will not be enough moisture to sprout seed. A drought in the fall hinders preparation of land for planting fall and winter crops and prevents germination of seed at the optimum time. At the same time, however, a dry fall is favorable for harvesting crops which mature at that time. (2) In many parts of the humid area, especially in the southeast half, crops are grown throughout the year. The moisture requirement varies with each crop and with the stage of development of the crop. A

TABLE 1.—DROUGHT PERIODS, 1949-1958

Station[a]	Number	Length, in Days	
		Average	Limits
(1)	62	24	14 - 75
(2)	56	22	14 - 59
(3)	75	23	14 - 59
(4)	58	23	14 - 72
(5)	61	23	14 - 59
(6)	59	23	14 - 68
(7)	63	23	14 - 81
(8)	65	24	14 - 54

[a] Stations: (1) Moline; (2) Mt. Carmel; (3) Ft. Smith; (4) Memphis; (5) Alexandria; (6) Selma; (7) Albany; and (8) Columbia.

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[3] "Weather Facts as Related to the Place of Irrigation in Cotton Production in the Mid-South," by A. L. King, U. S. Weather Bur., Dept. of Interior, Washington, D. C., a mimeographed paper presented July, 1954 at the Eighth Annual Beltwide Cotton Mechanization Conf., Little Rock, Ark.

[4] "Weather Facts, Drought Days, and Supplemental Irrigations," by A. L. King, U. S. Weather Bur., Dept. of Interior, Washington, D. C., October, 1954, a mimeographed paper presented at several irrigation workshops.

drought in the planting period can be disastrous; one in the period of maximum growth and fruiting will reduce yield materially; one in the harvest season, however, may be necessary or at least highly desirable.

Table 2 summarizes the number of drought days experienced in the 10-yr period under study. The average monthly number is about 7 from February through April and about 10 from May through July. The number increases rapidly from 14 in August to a peak of about 21 in October, then decreases to about 15 in November and 12 in December and January. In the northern portion of the humid area, or in any area where no crops are grown during the winter months, the number of drought days during the "no-crop" period may be disregarded except for academic reasons.

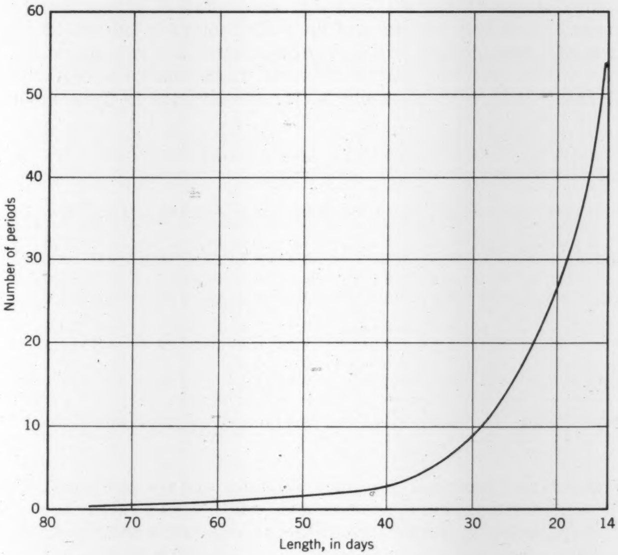

FIG. 1.—DROUGHT PERIOD FREQUENCY, 1949-1958

The frequency of exhaustion of moisture supplies during the growing season and the attendant water deficits have been determined for nine southern states by C. H. M. van Bavel. A summary of the method used and the results obtained is given elsewhere.[5]

SUPPLEMENTAL IRRIGATION

Knowing that droughts occur, the farmer now investigates the possibility of overcoming them. He finds that supplemental irrigation, if economically feasible in his operations, will provide the moisture needed by his crops.

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[5] "Water Deficits and Irrigation Requirements in the Southern United States," by C. H. M. van Bavel, Journal of Geophysical Research, October, 1959.

Supplemental irrigation is used to keep soil moisture above a predetermined minimum and at such a level that the crop will grow at the optimum rate. Without irrigation, antecedent conditions over a considerable period of time play a most important role. With irrigation, the role is minor or nonexistent, since the ground is kept in prime condition at all times, in winter as well as in summer. In the warmer sections of the humid area, both fall and winter crops are grown. They can be irrigated if necessary, while water can be withheld from those crops in which harvesting operations can best be carried out with a drier soil.

Various approaches have been made to the problem of trying to determine when to irrigate. Basically, all methods must balance the soil moisture loss with water added to the soil either from rainfall or irrigation. The problem is almost infinitely variable, but the main factors to be considered are surface runoff, underground drainage, evaporation, and transpiration. The mean monthly and annual evaporation amounts from some Class A stations in the humid area[6] are given in Table 3. The evaporation is from a water surface under standard conditions for such stations and are given to show the magnitude of evaporation. Some researchers use 0.7 as a factor to convert open-water evaporation to values applicable to vegetative surfaces. The following comparison gives some indication of the evaporation from a water surface as compared with that from soil. C. S. Slichter found that evaporation from a water surface in slightly less than one certain month was 10.90 in. From cultivated soil, with the water table one foot below the surface, it was 4.88 in., and from uncultivated soil, it was 5.83 in. Evaporation from soil with a capillary lift of 2 ft was 2.23 in., and for a lift of 3 ft, 0.80 in.[7] One principal effect of drainage is to reduce evaporation. Drainage presupposes an excess of moisture on the surface and in the upper few feet of soil, the layer from which evaporation takes place. By removal of surface water and by taking away gravity water from the upper layer of soil, drainage materially reduces the opportunity for evaporation.

TABLE 2.—AVERAGE MONTHLY AND ANNUAL DROUGHT DAYS, 1949-1958

STATION	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Moline	15.1	10.6	11.3	7.8	9.3	2.6	5.3	11.5	17.2	19.6	21.8	19.3	151.4
Mt. Carmel	8.2	8.8	6.1	7.3	6.4	5.4	8.3	15.3	16.9	19.0	9.8	11.0	122.5
Ft. Smith	15.3	9.8	10.4	8.3	7.6	16.1	16.6	15.7	17.5	19.9	17.0	19.5	173.7
Memphis	7.5	2.0	4.4	5.5	13.1	10.6	10.4	18.1	17.1	19.9	11.0	15.7	135.3
Alexandria	8.1	5.3	5.6	6.0	13.1	12.6	12.6	19.9	20.4	19.9	10.8	8.4	142.7
Selma	12.7	8.2	4.5	4.5	12.7	8.5	6.8	16.3	20.5	24.8	14.3	4.8	138.6
Albany	17.3	7.6	7.3	8.0	15.8	9.7	6.8	9.7	13.6	20.7	19.1	11.3	146.9
Columbia	7.8	7.6	3.9	12.4	17.5	9.5	14.3	9.0	16.2	25.1	16.8	12.9	153.0
Mean	11.5	7.5	6.7	7.5	11.9	9.4	10.1	14.4	17.4	21.1	15.1	12.9	145.5

If water for irrigation is to be obtained from a reservoir, the engineer must take into account evaporation from the reservoir. Data obtained from

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[6] "Mean Monthly and Annual Evaporation from Free Water Surface," Tech. Paper No. 13, U. S. Weather Bur., Dept. of Interior, Washington, D. C., July, 1950.

[7] Elements of Hydrology, by A. F. Meyer, John Wiley and Sons, Inc., New York, 1928.

TABLE 3.—MEAN MONTHLY AND ANNUAL EVAPORATION (INCHES), CLASS A STATIONS

Station	Years of record	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
Springfield, Ill.	8	. . .	. . .	. . .	5.19	5.90	6.98	8.39	7.22	5.67	3.72	1.97	. . .	. . .
Hope, Ark.	12	2.34	2.66	4.38	5.54	6.41	7.10	8.05	7.63	5.97	4.68	2.67	1.97	59.48
Hackberry, La.	10	3.53	3.64	4.78	6.32	8.09	7.82	8.62	8.04	7.02	5.98	3.78	2.89	70.51
Crowley, La. (BPI)	39	2.49	2.68	3.89	4.80	5.92	6.19	5.86	5.74	5.12	4.43	3.23	2.40	52.75
Vicksburg, Miss.	7	1.67	2.10	3.79	4.96	5.95	6.60	7.13	6.68	5.06	3.91	2.34	1.42	51.61
Fairhope, Ala.	14	1.98	2.36	3.64	4.91	6.12	6.20	5.88	5.61	4.53	3.68	2.37	1.62	48.90
Orlando, Fla.	7	2.88	3.78	5.35	6.56	8.00	6.87	6.83	6.06	5.14	4.82	3.43	2.67	62.39
Experiment, Ga.	12	2.08	2.54	4.39	5.81	7.34	7.76	7.03	6.28	5.42	4.21	2.58	1.95	57.39
Murphy, N. C.	14	1.03	1.49	3.00	4.32	5.54	5.88	5.74	5.07	4.10	2.89	1.64	0.90	41.60

TABLE 4.—EXAMPLE OF DEPLETION METHOD USED IN

(A)	1.50						2.25					
(B)	.10		.10		.10		.15		.15		.20	
Month	Jan.		Feb.		Mar.		Apr.		May		June	
Day	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
1		100	.07	88		97	1.41	0	.19	90	.88	0
2		110	.19	79		107		15	.73	32	.65	0
3	.15	105	.02	87	1.76	0		30	.03	44	.98	0
4	.04	111		97		10	1.17	0	T	59	.10	10
5		121		107	.02	18	.28	0		74		30
6		131		117		28		15		89	.38	12
7		141		127	T	38	.04	26		104		32
8	XXX			137		48		41		119		52
9		10		147		58		56	1.25	9		72
10		20	XXX		T	68		71	.01	23		92
11		30		10	.10	68	1.10	0	T	38		112
12		40	.06	14	1.39	0		15		53		132
13		50		24		10		30		68	T	152
14		60		34		20	.03	42		83		172
15		70		44		30		57		98		192
16	T	80	1.68	0		40	.70	2		113	XXX	
17		90		10	.19	31		17		128		20
18		100	1.77	0	1.00	0		32		143		40
19		110	T	10		10		47		158		60
20	.07	113		20		20		62		173	.26	54
21		123		30	1.55	0	.05	72		188	.26	48
22	.48	85		40		10	T	87		203	.03	65
23		95	.06	44	1.78	0	T	102	.20	198		85
24	.30	75	T	54	.67	0		117	.53	160	1.40	0
25	.21	64	.07	57		10		132	T	175		20
26	.10	64		67		20		147	.17	173		40
27	.05	69		77		30	.10	152		188	.89	0
28	.07	72		87	T	40	.22	145		203	3.25	0
29		82				50	.24	136		218		20
30		92				60	.57	94	???			40
31	.17	85			2.13	0			.08	7		

evaporation stations throughout the United States has been summarized[8]. Of special interest are (1) a table giving average annual, or seasonal, evaporation at 78 Class A stations, and (2) maps of the United States showing (a) average annual Class A pan evaporation in inches; (b) average annual lake evaporation in inches; (c) average annual Class A pan coefficient in percent; (d) average May-October evaporation in percentage of annual; and (e) standard deviation of annual Class A pan evaporation in inches.

Evapotranspiration, the end product of a large number of weather and plant factors, has been the object of study by many researchers, among them being Thornthwaite, Penman, van Bavel, and many others.

A simplified and streamlined method, the depletion method,[9] may be used easily by any individual to determine the approximate time to irrigate. A

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[8] "Evaporation Maps for the United States," Tech. Paper No. 37, U. S. Weather Bur., Dept. of Interior, Washington, D. C., 1959.

[9] First presented by A. L. King at the Fifth Annual Sprinkler Irrigation Meeting held at Virginia Poly. Inst., Blacksburg, Va., March, 1956.

DETERMINATION OF TIME OF IRRIGATIONS, ALEXANDRIA, LA., 1957

2.00						2.25				1.50	
.20		.20		.20		.15		.15		.10	
July		Aug.		Sep.		Oct.		Nov.		Dec.	
(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
	60	.02	57	.05	15		147		129		96
	80		77	.31	4	.05	157		144		106
.03	97		97	.94	0		172	.21	138	.05	111
	117		117		20		187	.11	142		121
	137		137		40		202		157		131
	157		157		60		217	1.99	0		141
	177		177		80	XXX		.77	0	1.36	15
	197		197	.05	95		15	2.25	0		25
XXX		.23	194		115		30		15		35
	20	XXX			135		45		30		45
	40	T	20		155		60		45		55
	60		40	1.48	27		75	.10	50		65
	80		60		47		90	3.38	0		75
	100		80		67		105	.40	0	.25	60
	120		100		87	.52	68	.03	12	.06	64
	140		120	.30	77	3.19	0		27		74
	160		140	.21	76		15	.36	6		84
.20	160		160		96		30	1.26	0		94
1.62	18	.04	176		116		45		15	1.70	0
.29	9		196	.47	89		60		30		10
.08	21	XXX			109	.45	30	.40	5		20
.47	0		20	.15	114	1.07	0	1.30	0		30
	20		40	.16	118	3.66	0	.22	0		40
	40		60		138		15	.80	0	.83	50
1.46	0		80		158		30		15		0
	20		100	1.20	58		45		30	.02	8
	40		120	.06	72		60		45	.24	0
	60		140		92		75	.04	56	.89	0
	80		160		112		90		71		10
.11	89		180		132	.06	99		86		20
.70	39	???					114			.09	21

record of the daily rainfall and a running moisture-depletion total, which is assumed to include both evapotranspiration and underground drainage, should be kept. In the Mid-South, the method has been used at 14 stations for more than 15 yr and the times of irrigations indicated by the method have agreed very closely with the times of actual irrigations. This fact tends to prove that the assumptions made are basically correct.

Table 4 is given as an example of the application of the depletion method in determining when to irrigate. Average soil and weather conditions other than rainfall must be assumed to set up specific factors to be used. Daily precipitation amounts at Alexandria, La., for 1957[2] are entered under the columns (1) in Table 4. An assumed average daily moisture-depletion amount during each month is entered in line (B); these amounts vary from 0.10 in. per day in the cooler months to 0.20 in. per day in summer. After taking rainfall into consideration, the daily cumulative depletion is given under the columns (2). When the cumulative depletion reaches the total given in line

TABLE 5.—TOTAL NUMBER OF MONTHLY AND

Station[b]	Jan.		Feb.		Mar.		Apr.		May		June	
(1)	8	(2)	8	(1)	9	-	6	-	9	(1)	8	(5)
(2)	4	(2)	3	-	2	(1)	3	(1)	5	(1)	10	(2)
(3)	9	-	4	-	6	-	4	-	6	(1)	17	(1)
(4)	1	(2)	2	-	2	-	2	(1)	8	-	12	(2)
(5)	3	(1)	3	-	3	(1)	4	-	4	(2)	17	-
(6)	5	(2)	4	-	2	-	2	(1)	10	(1)	9	(7)
(7)	9	-	5	-	5	-	4	(1)	6	(1)	12	(4)
(8)	7	-	6	(1)	1	-	7	(2)	9	-	20	(1)
Mean[c]	0.6	(.1)	0.4	([d])	0.4	([d])	0.4	(.1)	0.7	(.1)	1.3	(.3)

[a] Parentheses denote irrigations followed shortly by more or less significant rains.
[b] Stations: (1) Moline; (2) Mt. Carmel; (3) Ft. Smith; (4) Memphis; (5) Alex-
[c] Mean number of irrigations per station per month and year.
[d] Less than .05.

(A), an irrigation is indicated. The greatest allowable cumulative amounts vary from 1.50 in. in the cooler months to 2.00 in. in summer, and to 2.25 in. in the spring and fall. The date of an indicated irrigation is shown in Table 4 by the entry "XXX," as shown on June 16. The entry "???" is used to show that an irrigation is of doubtful value, because rain came shortly after that time, as shown on August 31. The cumulative entry on January 1 is 100, which indicates the deficiency carry-over from the last day of the previous month was 90 (0.90 in.). With no rain on January 1, 10 is added, giving the 100 entry; it is 110 on January 2. On January 3, the entry would have been 120 if no rain had fallen; 0.15 in. did fall, however, so the entry is 120 minus 15, or the 105 actual entry.

The depletion method was used for the entire 10-yr period of study at the eight selected stations. Results are given in Table 5. The total number of irrigations per month and year was calculated for each station. The irrigations of doubtful value are indicated in parentheses. There was not a single month in the year in which at least one irrigation was not required at some time during the 10-yr period. The average number of irrigations varied from about 0.4 per station per month in February, March, and April, to about 2.0 in August. On an annual basis, the average was about 12 per yr or about one irrigation per station per month. These irrigations were necessary to maintain the moisture content of the soil at an optimum level throughout the entire root zone.

There were 960 station months in the period under study. No irrigations were required in 263 (about 27.4%) of these months, an average of about 3.3 per station year; of this total, 176 (67%) were in the period January-April, inclusive, with only 17 (6.5%) being in the period June-September, the months when most summer crops have the maximum water requirements for optimum growth and fruiting. As a contrast, the number of irrigations required in the June-September period totalled 542, an average of about 1.7 per station per month. There were 286 months (about 30%) in the 10-yr period in which two or three irrigations were required. The greatest annual number of irrigations indicated at any station was 19 at Moline in 1956; the average at all stations, 16.5; the least number, 6 at Memphis in 1957; the

ANNUAL IRRIGATIONS, 1949 - 1958[a]

July		Aug.		Sep.		Oct.		Nov.		Dec.		Annual	
14	(2)	16	(3)	18	(3)	13	-	11	(1)	15	(2)	135	(20)
14	(2)	20	(2)	17	(2)	10	-	7	-	7	(3)	102	(16)
19	(1)	19	(2)	13	(5)	11	-	11	-	13	(1)	132	(11)
14	(1)	18	(3)	17	(2)	12	-	7	-	7	(1)	102	(12)
10	(1)	19	(1)	17	(2)	11	(1)	7	(2)	4	(4)	102	(15)
11	(2)	18	-	19	(2)	12	(1)	10	(1)	3	-	105	(17)
8	(2)	17	(2)	13	(2)	16	-	12	(1)	6	(1)	113	(14)
13	-	13	-	15	(1)	12	(1)	13	-	8	(2)	124	(8)
1.3	(.1)	1.8	(.2)	1.6	(.2)	1.2	(d)	1.0	(.1)	0.7	(.2)	11.4	(1.4)

andria; (6) Selma; (7) Albany; and (8) Columbia.

average at all stations, nine. Of the 1,028 irrigations indicated at all stations, 113 (about 11%) were of doubtful value because significant rains fell shortly after the times of irrigation. Adequate drainage is of prime importance in these 113 cases to prevent crop damage; they are included in the total number of times (1,039) significant surface drainage was needed, as shown in Table 11.

Much additional research is needed in the irrigation field. An article by H. H. Engelbrecht,[10] may help those who are interested in this phase of the overall problem.

RAINFALL AND DRAINAGE

The land is now protected by levees and flooding from streams is eliminated; lowlands and swampy places have been drained; crops are being grown. The farmer finds that crop yield is cut drastically in some years, and to some extent in almost every year, by droughts. He eliminates this uncertainity by supplying water during dry periods by supplemental irrigation. The land is kept in prime condition and crops grow rapidly. Just at the time it appears that a bumper crop is forthcoming, a heavy and prolonged rain falls; the crop is flooded, resulting in a total loss in some areas and severe damage in the remaining area. The farmer realizes, too late, that he has not taken into consideration all factors in the case. He has forgotten that heavy rains can be expected from time to time and that provision must be made to take care of surface runoff from the rains. The drainage need is greater if irrigation is practiced because the moisture content of the soil is maintained at a fairly high level, thus resulting in greater and faster runoff from heavy rains. If the field had not been irrigated prior to the rain, more water would have been absorbed by the soil, with less runoff resulting. The reduction would be less than might be expected, however, because it takes considerable time for moisture to soak deeply into the soil, except possibly in the case of a

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[10] "The Application of High-Speed Computers in Irrigation Research," by Howard H. Engelbrecht, Bulletin of the American Meteorological Society, November, 1959.

TABLE 6.—PRECIPITATION,

Station[a,b]		Jan.	Feb.	Mar.	Apr.	May	June	July
(1)	(A)	1.58	1.66	2.36	3.15	3.23	4.44	3.71
	(B)	3.56	2.72	5.06	5.87	5.90	7.13	5.60
	(C)	0.31	0.49	0.31	1.10	1.82	1.55	1.02
(2)	(A)	4.82	3.90	4.44	4.92	4.21	4.37	4.40
	(B)	13.47	8.72	7.52	8.42	7.75	8.10	12.78
	(C)	1.56	0.89	1.33	1.79	2.27	1.43	2.13
(3)	(A)	3.65	3.70	3.65	4.45	4.99	3.76	3.26
	(B)	11.33	7.94	8.52	10.32	12.09	10.40	9.50
	(C)	1.20	0.55	1.40	0.90	1.58	0.38	0.69
(4)	(A)	7.17	5.04	4.87	5.90	5.01	4.37	3.80
	(B)	15.45	9.58	9.23	12.06	10.57	10.30	6.62
	(C)	1.75	1.90	1.36	2.24	1.07	0.24	1.32
(5)	(A)	4.21	5.52	5.49	5.92	6.27	4.10	5.01
	(B)	8.00	9.45	10.59	10.43	16.90	9.08	8.08
	(C)	1.64	1.18	0.55	1.65	1.43	0.51	2.66
(6)	(A)	3.18	4.95	5.69	5.63	4.22	3.93	5.27
	(B)	5.02	8.46	9.91	9.64	8.53	5.23	11.24
	(C)	1.25	2.17	2.86	2.53	0.39	1.59	0.90
(7)	(A)	2.32	3.61	4.07	5.01	3.36	3.98	6.04
	(B)	4.77	6.49	6.10	8.07	6.32	7.69	9.98
	(C)	0.89	1.05	0.08	2.06	1.65	2.21	3.46
(8)	(A)	2.57	3.38	3.85	3.65	3.17	2.88	4.95
	(B)	4.90	6.38	7.00	5.89	6.71	6.44	11.79
	(C)	0.97	1.12	1.25	1.37	0.29	1.26	1.15

[a] Stations: (1) Moline; (2) Mt. Carmel; (3) Ft. Smith; (4) Memphis: (5) Alex-

[b] (A) Mean; (B) Greatest; and (C) Least.

TABLE 7.—GREATEST 24-HOUR PRECIPITATION,

Station	Years of record	Jan.	Feb.	Mar.	Apr.	May
Moline, Ill.	20	1.84	1.90	2.38	2.72	2.13
Mt. Carmel, Ill.	40	4.25	2.90	6.20	3.43	3.48
Ft. Smith, Ark.	68	5.42	5.56	3.83	4.98	5.80
Memphis, Tenn.	79	5.75	4.57	9.30	5.26	4.29
Alexandria, La.	53	6.16	5.20	5.40	7.00	5.46
Selma, Ala.	54	4.77	3.85	8.06	8.74	3.24
Albany, Ga.	67	4.30	4.60	6.39	7.60	4.91
Columbia, S. C.	63	2.93	4.23	2.83	3.98	4.88

1949 - 1958

Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Average Snow	Average Temperature
3.82	2.05	2.20	1.45	1.72	31.37	24.3	50.2
8.00	3.85	7.43	2.99	3.82	48.60		
1.05	0.49	0.04	0.60	0.50	20.20		
2.36	3.32	2.91	4.02	3.78	47.45	20.4	55.7
4.71	5.60	7.32	8.22	8.80	66.59		
0.10	0.85	0.97	0.81	0.54	32.63		
3.11	3.25	3.02	3.15	1.92	41.91	4.5	62.0
5.83	5.88	12.05	7.03	5.42	60.78		
1.42	0.06	0.57	0.59	0.53	30.57		
3.32	2.94	2.89	3.38	3.92	52.61	5.2	61.8
5.75	7.22	8.16	8.89	7.44	74.78		
1.46	0.28	0.66	0.69	1.72	34.79		
2.78	2.73	4.16	4.00	5.16	55.35	2.7	67.5
7.41	10.28	9.00	13.62	9.34	73.07		
0.10	0.34	T	0.36	1.76	36.69		
2.81	3.35	1.43	2.54	5.39	48.39	0.3	66.5
7.51	7.62	3.05	5.81	11.12	54.80		
0.53	0.24	0.43	0.51	2.27	30.00		
3.59	4.58	1.42	2.50	3.47	43.95	0.2	66.8
6.20	11.25	3.26	8.28	9.53	59.56		
0.73	0.70	0.22	0.40	0.64	32.62		
6.00	4.38	1.55	2.04	3.10	41.52	2.0	64.0
16.72	8.78	2.65	7.20	7.43	53.44		
1.27	0.76	0.32	0.58	0.32	27.38		

andria; (6) Selma; (7) Albany; and (8) Columbia.

SELECTED STATIONS

June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Greatest
3.43	3.45	4.44	4.77	2.39	1.66	3.38	4.77
3.34	4.54	3.70	5.60	5.30	3.66	3.57	6.20
8.58	3.90	5.10	4.36	3.54	4.19	6.40	8.58
9.67	5.42	4.55	4.66	6.44	10.48	5.40	10.48
21.40	9.75	3.85	6.51	7.40	6.08	5.65	21.40
6.10	4.87	5.35	4.30	4.69	4.07	6.40	8.74
4.15	5.10	4.90	6.84	3.10	4.33	4.03	7.60
4.13	5.00	7.40	5.50	3.30	2.16	3.31	7.40

TABLE 8.—GREATEST 24-HOUR PRECIPITATION FROM STATIONS

Area	Jan.	Feb.	Mar.	Apr.	May	June
Northern Ill.	3.65	3.53	3.80	5.60	6.00	6.86
Southern Ill.	6.09	5.50	7.16	6.21	5.90[a]	5.77
Western Ark.	8.00[a]	5.56	7.50[a]	11.40	9.01	12.00
Western Tenn.	8.52	4.84	9.30	5.35	5.54	9.67
Central La.	11.13	7.30	8.15	9.95	10.00[a]	21.40
Southern Ala.	9.98	9.00[a]	10.73	10.00	8.00[a]	12.40
Southern Ga.	8.23	6.30	10.88	9.00	7.30	7.00
Central S. C.	5.85	4.70[a]	5.50[a]	7.70[a]	7.50[a]	7.04

[a] Precipitation is interpolated from amounts in surrounding areas.

sandy soil. The farmer now realizes that he must provide adequate drainage as one of the final steps in his effort to provide insurance for optimum crop yields. In planning a drainage system, it is a duty of the engineer to point out to the farmer that it is highly desirable to level the land (1) to eliminate the "low spots" which exist in so-called level fields, and (2) to establish a predetermined slope or grade to eliminate most of the effects of sheet erosion. Water from rainfall will then soak much more evenly into all portions of the field, and when irrigation is practiced, one portion of the field will not be too dry while the other portion is too wet. In addition, more rainfall will soak into the soil of a properly levelled field, with a reduction in the number of irrigations required and also in erosion of the topsoil. The capital expenditure for drainage is necessary, especially so when irrigation is practiced. A detailed study of rainfall in the humid area shows without question the need for drainage.

For the 10-yr period 1949-1958, Table 6 shows (1) the mean monthly and annual; (2) the greatest monthly and annual; and (3) the least monthly and annual precipitation[2] at the eight stations used in this study. The mean monthly amount is 3.78 in., which is almost adequate for most crops in most cases if it fell at the right times. The mean of the greatest monthly amounts, 8.08 in., is 214% and the mean of the least monthly amounts, 1.05 in., is 28% of the actual mean monthly precipitation amounts. Many months in the 10-yr period had so much total rain that crops were damaged and drainage was needed, while, there were also many other months with not enough total rain to sustain any crop. The average annual snowfall and temperature are also given in Table 6. At the eight stations under study, every month had a mean temperature above freezing except for two months in mid-winter at Moline and for one month at Mt. Carmel.

Records of mean annual precipitation, or even the monthly amounts in most cases, are not of much value in the design of most works for the utilization or control of water. Runoff resulting from the average precipitation can seldom be utilized. Records of exceptional conditions are of much greater importance than records of average conditions. In drainage problems, we should consider the occurrence of rains of sufficient amount and intensity to produce material runoff and to require adequate drainage. Light rains are of little significance because of the small amount of moisture involved and

WITH AT LEAST 10 YEARS' RECORD THROUGH 1949

July	Aug.	Sep.	Oct.	Nov.	Dec.	Greatest
6.53	9.15	9.08	5.20	5.00[a]	3.62	9.15
7.50[a]	9.50[a]	8.08	7.99	5.06	5.15	9.50[a]
8.00[a]	10.03	8.92	9.00[a]	6.47	7.00[a]	12.00
7.02	6.80	6.27	8.43	10.48	6.60	10.48
11.20	15.00[a]	11.50[a]	8.00[a]	12.50[a]	8.50[a]	21.40
12.00[a]	9.90[a]	10.25	8.03	11.20	10.15	12.40
9.90	12.00[a]	14.00[a]	9.00[a]	10.00[a]	6.08	14.00[a]
12.50[a]	11.00[a]	10.72	7.40	6.00[a]	6.00[a]	12.50[a]

TABLE 9.—COMPARISON BY HERSHFIELD AND WILSON

Storm Type	Rainfall, in inches			
	1-hr	3-hr	24-hr	48-hr
(1) Tropical	1.84	4.75	13.66	16.65
(2) Non-Tropical	2.34	6.40	14.01	14.01

also because vegetation intercepts a considerable portion of such rains.

It has been estimated that rainstorms from which we receive much of our excessive precipitation cover an area about 15 mi in diameter on the average. Precipitation falls in an irregular manner, with respect to time, in a given locality. Records for a single station furnish a far less satisfactory basis for a valid conclusion regarding the frequency of given rates of excessive precipitation than the records of several stations in a general area. If an area is meteorologically homogeneous, the records of several stations in a limited area may be combined, and they are virtually equivalent to a longer record at a single station, one record supplements the other.[11]

Table 7 gives the maximum 24-hr precipitation as recorded in each month for the period of record at the eight stations under study.[12] The length of record varies from 20 to 79 yr. The greatest 24-hr amounts range from 4.77 to 21.40 in., with an average of 9.40 in. for the eight stations. Table 8 gives similar amounts recorded at some station in the general area of the states where the eight stations are located. The record includes stations in those areas with 10 or more years of record, and the greatest amounts range from 9.15 to 21.40 in., with an average of 12.68 in., about 35% greater than the average at the eight point locations given in Table 7. The transposition of storms in a homogeneous area is standard practice in hydrometeorological work, and the engineer, as a matter of precaution, should refer to the maximum average depth of rainfall for selected time periods over areas of various

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[11] Applied Hydrology, by Linsley, Kohler, and Paulhus, McGraw-Hill Book Co., New York, 1949.

[12] "Maximum 24-Hour Precipitation in the United States," Tech. Paper No. 16, U. S. Weather Bur., Dept. of Interior, Washington, D. C., January, 1952.

TABLE 10.—GREATEST STATION PRECIPITATION

Station	Period	1		2	
Peoria, Ill.	1940 - 1950	2.23	1940	2.28	1940
	1905 - 1950	2.60	1931	3.18	1911
Evansville, Ind.	1940 - 1950	1.63	1946	2.23	1943
	1899 - 1950	2.79	1916	3.11	1916
Memphis, Tenn.	1940 - 1950	1.70	1948	2.51	1950
	1890 - 1950	3.25	1929	4.70	1929
Shreveport, La.	1940 - 1950	2.81	1940	4.02	1942
	1902 - 1950	3.15	1908	5.19	1905
Meridian, Miss.	1940 - 1950	2.54	1941	2.73	1940
	1899 - 1950	3.66	1906	3.77	1906
Apalachicola, Fla.	1940 - 1950	3.33	1946	5.01	1946
	1922 - 1950[a]	3.33	1946	5.01	1946
Macon, Ga.	1940 - 1950	2.89	1949	3.07	1941
	1899 - 1950	4.28	1923	6.55	1923
Columbia, S. C.	1940 - 1950	2.10	1949	2.57	1949
	1901 - 1950	2.28	1914	3.25	1911

[a] Record missing 1934 - 1936.

TABLE 11.—NUMBER OF TIMES SIGNIFICANT

Station	Jan.	Feb.	Mar.	Apr.	May	June
Moline	6	2	4	8	7	15
Mt. Carmel	15	11	14	13	12	13
Ft. Smith	10	13	9	14	13	11
Memphis	22	17	14	14	14	15
Alexandria	15	18	18	17	18	13
Selma	10	16	14	16	10	13
Albany	9	11	13	17	8	17
Columbia	7	10	12	9	9	7
Mean	12	12	12	14	11	13

sizes for all major storms in the United States.[13] If this is done by the engineer in planning drainage facilities, he will be able to consider the heaviest rainfall which can normally be expected in the general area of the project, along with such other items as factor of safety and economic feasibility.

There may be some question in the engineer's mind about the amount of rainfall in coastal areas in connection with tropical as compared with that from non-tropical storms. A comparison made in 1959 by Hershfield and Wilson[14] for such rains in New Orleans is shown in Table 9. These rains occurred (1) in October, 1937 and (2) in April, 1927. It was concluded that there

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[13] "Storm Rainfall in the United States—Depth-Area-Duration Data," Corps of Engrs., U. S. Army, 1945, (see additions published in later years).

[14] Hydrologic Services Div., U. S. Weather Bur., Dept. of Interior, Washington, D. C., November, 1958.

FOR 1, 2, 3, 6, 12, AND 24 HOURS

Hours							
3		6		12		24	
3.02	1947	3.10	1947	3.54	1950	5.06	1950
3.48	1927	4.33	1927	4.69	1915	5.52	1927
2.23	1943	3.03	1945	4.47	1943	5.15	1943
3.41	1924	4.40	1924	4.82	1910	6.94	1910
2.75	1949	3.70	1947	4.75	1949	5.26	1949
5.00	1929	7.03	1934	9.67	1934	10.48	1934
4.76	1942	4.92	1942	5.54	1949	6.83	1949
6.49	1905	7.54	1905	8.52	1933	12.44	1933
2.96	1942	3.94	1942	4.80	1942	4.84	1942
4.31	1936	5.49	1936	6.73	1936	9.50	1900
7.20	1946	8.97	1946	9.37	1946	10.06	1946
7.20	1946	8.97	1946	9.37	1946	11.71	1932
3.09	1941	3.58	1943	3.90	1943	5.55	1943
6.60	1923	6.71	1923	7.92	1928	8.36	1928
2.99	1949	4.52	1949	6.77	1949	7.40	1949
4.08	1911	4.52	1949	6.77	1949	7.40	1949

SURFACE DRAINAGE NEEDED, 1949 - 1958

July	Aug.	Sep.	Oct.	Nov.	Dec.	Annual
9	8	4	4	4	4	75
13	7	9	9	11	13	140
7	8	12	7	8	3	115
12	11	6	6	9	12	152
18	5	6	11	12	19	170
15	5	11	4	8	14	136
19	10	11	2	6	14	137
12	18	14	3	4	9	114
13	9	9	6	8	11	130

is no significant difference in tropical and non-tropical storm rainfall amounts.

While the total amount of rainfall must be considered, the intensity of its fall is equally important and is of prime concern to the engineer. As an aid to him, depth-duration curves are given to serve as a guide in planning drainage facilities. Two curves for the southeastern United States[15] show (1) Fig. 2, the average percentage of 24-hr rainfall which falls in hourly increments, and (2) Fig. 3, the average of 7-day precipitation which falls in daily increments. In 1-day rains, about 65% falls in 6 hr, 85% in 12 hr, and 94% in 18 hr. In 7-day rains, it is found that about 57% falls in 1 day, 74½ % in 2 days, 87% in 3 days, 92% in 4 days, 95% in 5 days, and 97½ % in 6 days.

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[15] "Rainfall Intensity-Frequency Regime, Part 2—Southeastern United States," Tech. Paper No. 29, U. S. Weather Bur., Dept. of Interior, Washington, D. C., March, 1958.

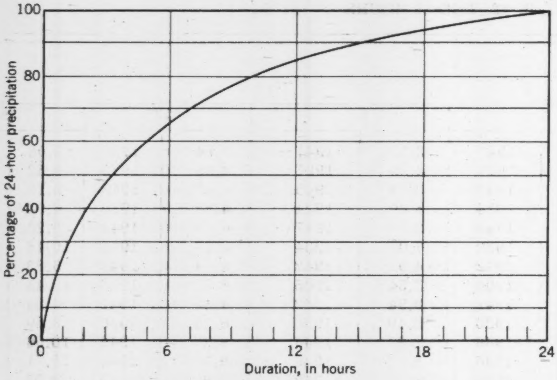

FIG. 2.—PERCENTAGE DEPTH-DURATION CURVE FOR 24-HR PRECIPITATION, SOUTHEASTERN UNITED STATES

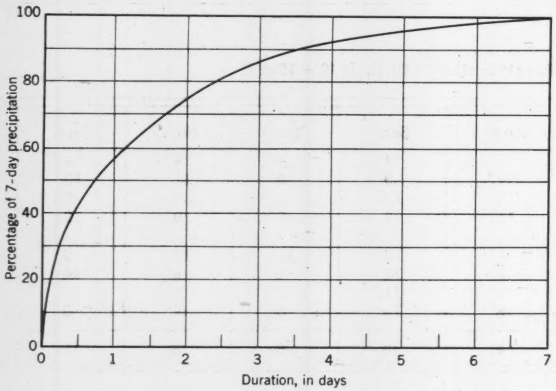

FIG. 3.—PERCENTAGE DEPTH-DURATION CURVE FOR 7-DAY PRECIPITATION, SOUTHEASTERN UNITED STATES

Runoff increases rapidly with an increase in the rate of fall of precipitation in short periods, and Table 10 gives some indication of the rates to be expected. Maximum amounts for 1, 2, 3, 6, 12, and 24 hr are given for the 10-yr period 1940-1950, and also for the entire period of record (averaging about 50 yr) at eight locations in the humid area.[16] The choice of stations is necessarily limited to "first-order" Weather Bureau stations at which an

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[16] "Maximum Station Precipitation for 1, 2, 3, 6, 12, and 24 Hours," Tech. Paper No. 15, U. S. Weather Bur., Dept. of Interior, Washington, D. C., various parts, 1954 to 1958.

autographic record of precipitation is available for a long period. It will be noted that in only eight of the 48 sets of comparative amounts in the table are the 10-yr record amounts the same as in the much longer records. In the remaining 40 sets, the longer-period amounts are about 8 to 113% larger than the corresponding 10-yr amounts. A comparison of the greatest 24-hr amounts in Table 10 with those in Table 8 again shows the need for considering a larger-than-recorded "station" rainfall for shorter periods, as the "area" rainfall for those periods is considerably greater in most cases. The depth-duration curves in Figs. 2 and 3 will give some indication of the proper time distribution of rains of greater amount than those shown in the tables.

Now that we know that heavy rain can be expected from time to time in the humid area, it may be well to know how many times significant surface drainage in an irrigated area is indicated. An inspection was made of the daily rainfall record at the eight stations under study. Consideration was given the time of year, antecedent conditions, and the particular pattern of rainfall. A rain of 1.50 in., for instance, in January might result in considerable runoff, while such a rain in August might give little or no runoff unless it came soon after a field had been irrigated. Table 11 indicates that considerable runoff may be expected 13 times each year (about once every four weeks) on the average at each station. The monthly average varies from 1.4 in April to 0.6 times in October; the annual average, from 7.5 at Moline to 17.0 times at Alexandria. In the critical growing period, April-August, inclusive, the average is 6.0 times per station; in the harvest period, September-November, inclusive, it is 2.3 times, a favorable value when a dry fall is needed. In examining published daily rainfall records, it should be remembered that the amounts are for 24-hr periods (for instance, midnight to midnight, 7 a.m. to 7.a.m., etc.), and that rains of 0.75 and 0.90 in., for instance, measured on successive days could well be one continuous rain of 1.65 in. in 24 hr or less, with considerable runoff.

A tabulation was made of the number of times when 24-hr rains of three or more, two or more, and one or more inches were measured at the eight stations under study. There were 70, 236, and 1,036 such occurrences, respectively, averaging about 0.9, 3.0, and 13.0 times per station year. Adequate drainage is needed, to prevent crop damage, in many of these cases.

CONCLUSIONS

Even in humid areas, all sections experience droughts, and supplemental irrigation may be used to advantage, if economically feasible and if an adequate water supply is available. Rain does not fall in many cases in proper amounts or at the times needed, although it may be adequate as far as total amount is concerned. Many rains fall in such amounts and at such excessive rates that adequate drainage for the protection of crops is required for optimum yields. It must be stressed that irrigation, without adequate drainage, may be a detriment rather than an aid. Irrigation, if practiced throughout the year, keeps the soil in optimum condition and permits the growth of suitable crops in all seasons, thereby raising the standard of living in humid areas. The very rapid increase in world population makes it necessary that we have a similar increase in food production and the logical solution of the problem is to increase production in the most fertile areas near population centers.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3220

CLIMATE AND CROPS IN HUMID AREAS

By J. A. Riley[1] and P. H. Grissom[2]

SYNOPSIS

Climatic influence must be considered in designing an engineering procedure to increase farm efficiency. Climatic influences that are considered to be of importance in humid areas are discussed herein: (1) the geographical variation of precipitation, temperature, sunlight and day-length, humidity, evaporation and evapotranspiration and their general effect on crops; (2) the microclimate and its effect on crops; (3) climatic influence on the secondary effects of crop production including: diseases, insects, dust and spray operations, tillage and weed control, and fertilization; and (4) specific weather relations of certain crops.

INTRODUCTION

The earliest agriculture in the United States was largely confined to humid regions. Farming was a simple matter of cutting down the forests and planting a crop in the cleared area. Crops were adequate because the soil was fertile, rainfall abundant, and economic pressures small.

Farming today is much different. In many cases the land has been misused and it is necessary to apply special practices to overcome this handicap. In nearly all cases, farming is more competitive and the farmer must take advantage of scientific advances to bring his operation up to a high degree of efficiency.

Note.—Published essentially as printed here, in December, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2685. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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Climate affects crop production directly by influencing plant development and indirectly by influencing production practices. An engineering procedure, designed to increase farm efficiency, must take into account these climatic influences.

CLIMATE OF THE HUMID AREA AND ITS EFFECT ON CROPS

The problem of climate's influence on crops is similar to most engineering endeavors, in that the climate must be measured in precise enough terms to determine its effect on crops, and yet stay in the realm of practicality. In this attempt, the general subjects precipitation, temperature, sunlight and day-length, humidity, evaporation, and evapotranspiration will be considered. (Data for particular cities or areas may be found in the United States weather bureau (USWB) publication "Climatological Data" available at most Weather Bureau offices.) A number of other climatic variables influence crop growth, however, these six are considered of primary importance. (Storm damage is an infrequent but important weather variable. Each year severe storms ruin crops in local areas, however as there is little or no control over this possibility, storm damage will not be considered in this report.)

For most agricultural purposes, the basic classification of climate is moisture distribution and the only measure of this distribution with a long period of record is rainfall. Many research works define a humid climate as one with over 30 in. of annual precipitation. Fig. 1 shows this classification includes most of the eastern third of the contiguous United States with scattered areas in the Far West (1).[3] To simplify the discussion, and to make the classification along familiar geographical lines, this paper will consider the humid areas to be: Minnesota, Iowa, eastern Kansas, eastern Oklahoma, eastern Texas and all the states to the east; also western Washington, western Oregon, and northern California.

Precipitation.—The largest average yearly precipitation amounts, as shown by Fig. 1, occur along the Pacific northwest coast and along the Gulf coast. In the northwest, precipitation drops off sharply 100 to 200 mi inland. Over the eastern half of the country, the decrease in precipitation away from the Gulf is much more gradual. Fig. 2 shows another measure of moisture distribution, the annual number of days with 0.01 in. of moisture or more (1). This distribution gives quite a different picture as to which areas are moist. For example, upper Michigan, which has only about 30 in. of annual precipitation, has approximately 160 days on which 0.01 in. of moisture is recorded.

The seasonal variation in precipitation is probably more important to agriculture than the total amount. The northcentral states have a summer maximum while a spring maximum is general in the South except along the east coast where a fall maximum reflects the infrequent but heavy rains of hurricanes. The northeastern states have very little season variation. Along the west coast, precipitation has a very strong winter maximum, quite different than the rest of the humid area.

Even after the precipitation distribution has been determined for a region by a system of averages, only part of the variation is known. Fig. 3 shows the average yearly precipitation for the driest 10 yr out of 40 (1). Compared

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[3] Numerals in parentheses refer to corresponding items in the Appendix.

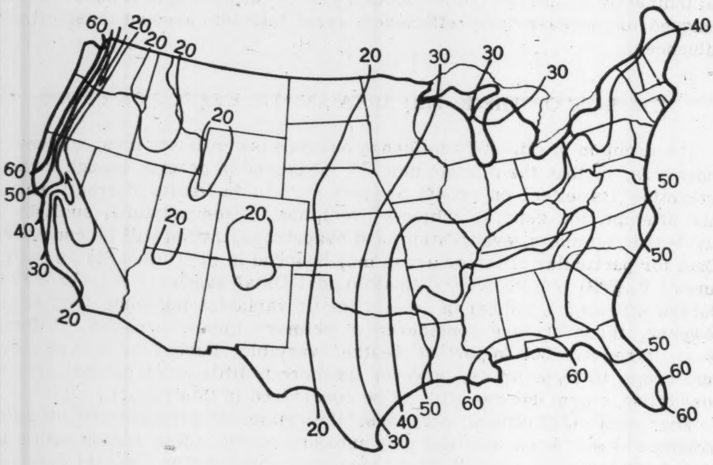
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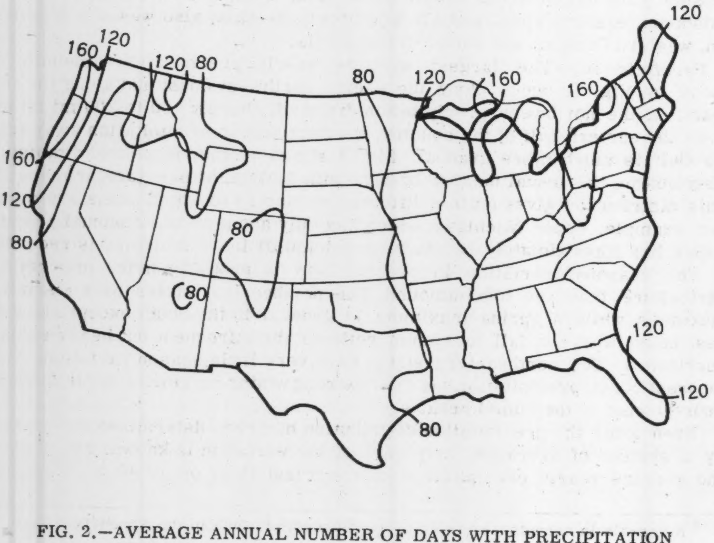
FIG. 1.—AVERAGE ANNUAL PRECIPITATION, IN INCHES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80

FIG. 2.—AVERAGE ANNUAL NUMBER OF DAYS WITH PRECIPITATION OF 0.01 IN. OR MORE

with Fig. 1, the area with 30 in. of annual precipitation is much smaller. When shorter periods of time are considered, even this wet area experiences significant droughts (2).

Very heavy rains that fall during short periods of time are most likely along the Gulf coast. Rainfall amounts of 12 in. to 15 in. in 24 hr have a reoccurrence interval of 100 yr along the coast and inland about 100 mi. These heavy downpours are much less likely further inland. From eastern Kansas through northern Illinois to New England, 24-hr rainfall amounts of 5 in. to 7 in. have the same reoccurrence value, whereas in northern Minnesota, the heaviest 24-hr rain likely to occur once in 100 yr drops to 4 in. to 5 in.

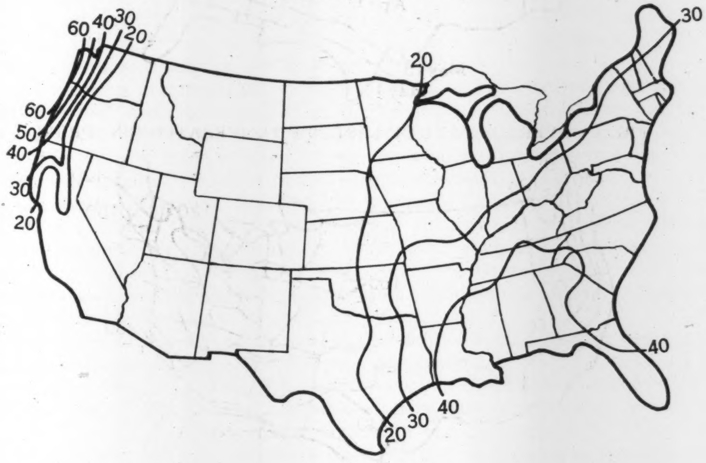

FIG. 3.—AVERAGE PRECIPITATION, IN INCHES, FOR THE 10 DRIEST YEARS IN 40 YEARS

Effect of Precipitation on Crops.—Precipitation is not essential for crop growth, however, an adequate supply of moisture is necessary and the cheapest and most common method of achieving this supply is by way of precipitation. More information on crop moisture requirements is found elsewhere in this paper.

Heavy local rains in short periods of time do great damage to crops. Directly, they can injure the crop by beating it down and indirectly by flooding. The flood potential of some crop land is greater than the rest because many high quality crops are grown near rivers on flood plains.

Temperature.—Temperature is the second major climatic classification. Fig. 4 shows the average date of the last 32° temperature in spring (3). Fig. 5 shows the average length of the growing season as determined by the interval between the average date of the last 32° temperature in spring and the first 32° temperature in fall. In the eastern humid region, the growing season de-

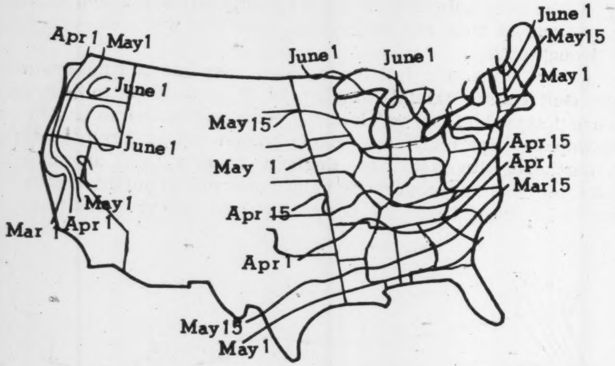
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FIG. 4.—AVERAGE DATE OF LAST 32°F TEMPERATURE IN SPRING

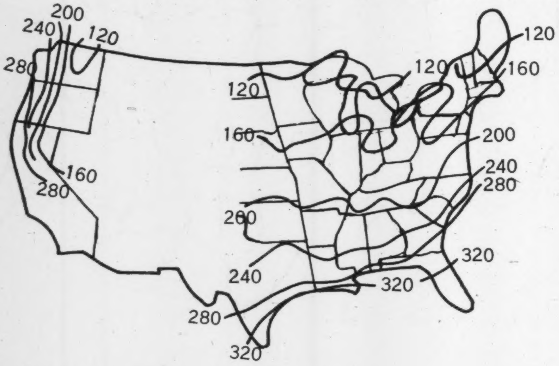
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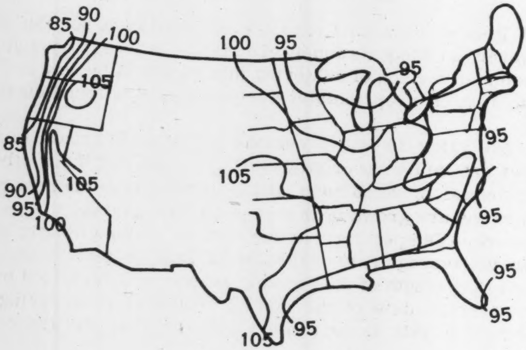
FIG. 5.—AVERAGE LENGTH OF GROWING SEASON, IN DAYS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95

FIG. 6.—AVERAGE ANNUAL MAXIMUM TEMPERATURE (°F)

creases from the southeast to northwest with minor increases around lakes. The growing season ranges from over 300 days along the Gulf coast to only about 100 days in some sections near the Canadian border.

Very high temperatures have lethal effects on some crops and Fig. 6 shows the average annual maximum temperature (1). The geographical variation of high temperature is very much smaller than the variation of the length of the growing season. Highest temperatures occur along the western edge of the eastern humid region and in the valleys of northern California.

Effect of Temperature on Crops.—Each crop has its own optimum, maximum and minimum temperature standards, however, most crops make their best development between 60°F and 90°F. Many plants make no growth when the temperature is down to 40°F whereas an extreme case, sorghum, practically stops growth when the temperature is down to 60°F. Depending on maturity and condition, most plants are killed by a temperature of 32°F or lower, and many others by 100°F or over.

The relation of temperature to crop production has evolved into two frequently quoted laws. According to A. D. Hopkin's Bioclimatic Law; starting in the sourthwest part of the country, such events as seeding time are generally delayed 4 days by each advance of one degree north latitude, five degrees of eastward longitude, and 400 ft of increased altitude (4).

Vant Hoff-Arrhenius' law for monomolecular chemical reactions holds true within normal temperature ranges and plant growth increases with each rise in temperature, approximately doubling for each 10°C increase. An extension of this law makes possible the "growing degree day" that is widely used by the vegetable packing industry as a guide for all phases of operation from the day of seedling to the final day of harvest.

The engineer has, for many years, utilized the degree day for heating requirement study and more recently as a measurement of air conditioning requirements. The "growing degree day" theory of plant development replaces growing time in days by accumulated heat units (5). For example, the pea is a cool weather plant and peas will germinate at a temperature of 40°F. Each degree of mean daily temperature over 40°F is a growing degree day. For sweet corn, 50°F is commonly accepted as the base. Variety requirements for the same crop vary, however, they are less than the difference between different crops. In the state of Wisconsin, the heat requirement for maturity varies from 1250 degree days for Alaskan peas to 1775 for Perfection peas. Some of the factors that influence the range of total heat units required for a crop or a variety are; soil type, slope, drainage, fertility level, depth of planting, spacing of plants, droughts, and vigor of seeds. The difference in day-length between northern and southern locations also influences the total, the longer the day the more rapid the development. This growing degree day procedure has great advantage to the user. It provides an effective basis for planning an uninterrupted supply of fresh crops at optimum maturity. It provides a means of determining labor and equipment needs. For the grower, it provides a measure of performance between different varieties and is a definite aid in quality control.

Sunlight and Day-length.—Sunlight measurement, as it affects growing plants, has been approximated by a number of different methods. USWB stations have rather long periods of record showing the average number of days of clear, partly cloudy and cloudy skies. There is a somewhat shorter period of record from many Weather Bureau stations showing the average percent of the daylight hours that the sun shines, for the average, during the summer months (Fig. 7) (1).

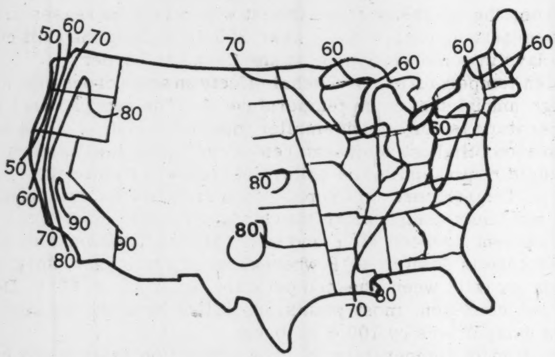
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FIG. 7.—PERCENTAGE OF POSSIBLE SUNSHINE, SUMMER (JUNE-AUGUST)

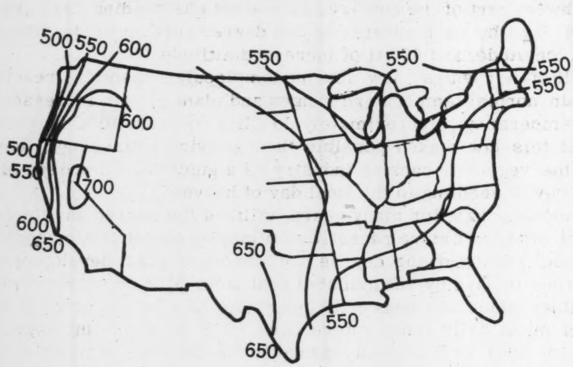
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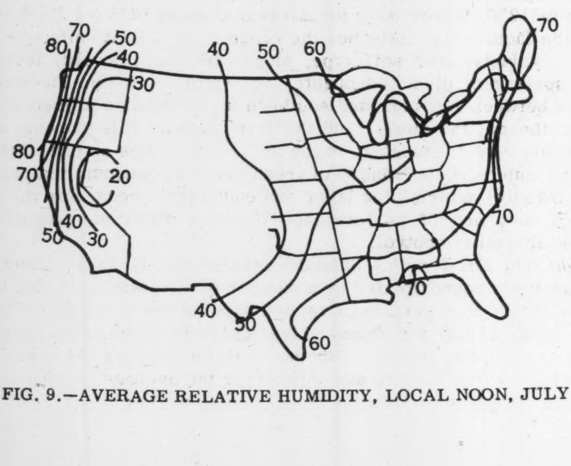
FIG. 8.—AVERAGE JULY SOLAR RADIATION, IN LANGLEYS PER DAY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60

FIG. 9.—AVERAGE RELATIVE HUMIDITY, LOCAL NOON, JULY

A more precise measurement of sunshine intensity is shown in Fig. 8 (6). It gives the 5-yr solar radiation average in Langleys per day for July for the period of 1953 through 1957. (A Langley is one gram calorie per square centimeter). More recently, measurements are being made of net radiation, which is the earth's outgoing radiation subtracted from the incoming radiation. Net radiation has the greatest use in plant growth determination, however it has great variability. It not only varies with geographical location, but in the same area, it varies because of different ground cover.

Day-length is a function of latitude. Along the northern border, day-length ranges from 8.2 hr in winter to 16.1 hr in summer. In southern Florida, the range is from 10.6 hr to 13.7 hr.

Effect of Sunlight and Day-Length on Crops.—Sunlight affects photosynthesis, the plant's production of food. Insufficient sunlight has a detrimental effect on crops. Too much sunlight for plant growth seldom, if ever, occurs except in cases where moisture is limited, and then the excess sunlight dehydrates a plant. Often, however, the bright sunshine of midday is not fully utilized by a plant and that percent of sunlight above a certain threshold is wasted.

The full potential of the greater amount of sunlight in the South offers opportunity for improved farm practice. One method that has been practiced is double cropping. Oats seeded in the fall and harvested in the spring are often followed by sorghum, corn, or soybeans. Oats have declined in their cash value, and double cuttings of sorghum are being tried as a method of utilizing a greater part of the incoming solar radiation in the South.

Day-length is important in plant maturity. The long summer days at high latitudes cause certain plants to develop and mature in a relatively short period of time. Photoperiodism is the process of day-length's effect on the life process of many plants. Some crops require long days to produce flowers, but increase in vegetative growth when days are short. Small grains, except rice, are long-day plants and flower in the long days of early summer. Soybeans generally delay flowering and maturity until days become short. Cotton and some other crops are not materially affected by this process.

Humidity.—The most commonly used measure of atmospheric moisture is relative humidity. Relative humidity is the ratio of the actual amount of moisture in the air to the amount of moisture that the air could hold if it were saturated. Fig. 9 shows the average noon July relative humidity (1). It is highest near large bodies of water and lowest in the large continental land mass of the country. Temperature influences relative humidity; warm air can hold more moisture than cold air. For example, if the relative humidity is 100% on a summer morning with a temperature of 65°, it will drop to around 40% in the afternoon if the temperature rises to 95°. This is accomplished without the removal of any moisture.

Wet-bulb temperature is a more conservative measure of moisture if temperature changes are involved. The wet-bulb temperature is the temperature that will be reached if water is evaporated from the bulb of a thermometer. When the actual content of atmospheric moisture is needed instead of an estimate of saturation, the wet bulb should be used instead of the relative humidity. Wet bulb temperature does not have the diurnal fluctuation that relative humidity has, also, it is more regular in geographical distribution. In contrast to the irregular lines on Fig. 9, the average July noon relative humidity, the average July wet bulb temperature decreases gradually from 75°F along the Gulf coast to 60°F along the northern border. In the western humid area, the average July wet-bulb temperature is 55°F to 60°F.

Effect of Humidity on Crops.—Humidity's effect on crop growth is mainly indirect. High humidity limits evaporation and transpiration and is quite helpful when the moisture supply is critical. High humidity, accompanied by clear skies and light winds at night is associated with heavy dew formation. Some researchers have found that the addition of dew is a significant addition to the moisture supply (7). High humidity promotes many plant diseases and has a lesser affect in promoting certain types of insects.

Evaporation and Evapotranspiration.—Evaporation increases with increases in temperature, sunshine, and wind, and with decreases in relative humidity or wet-bulb temperature. Various types of pans and porous cups have been used in an attempt to measure this variable weather element, but none gives results satisfactory to all researchers. Fig. 10 shows the average annual evaporation as determined from the USWB class A pan (8). Highest values of evaporation are found in the warm, drier, southwest part of the humid region, decreasing to the north and east in the cooler and moister areas.

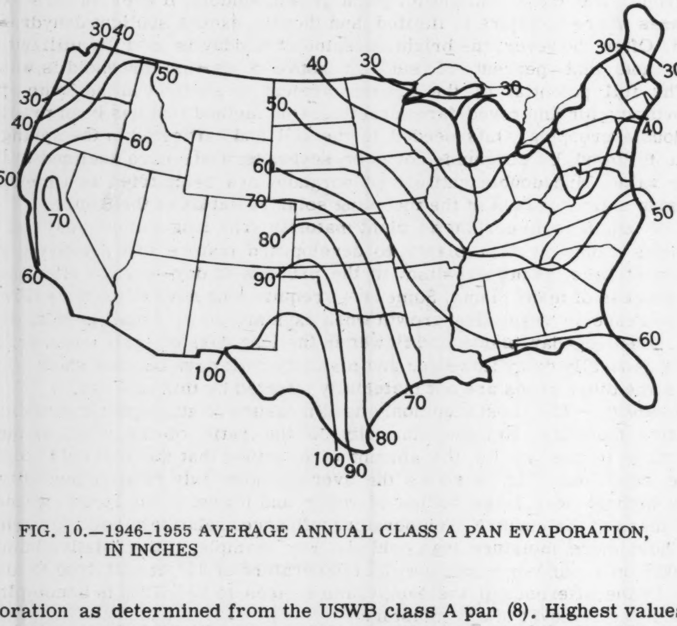

FIG. 10.—1946-1955 AVERAGE ANNUAL CLASS A PAN EVAPORATION, IN INCHES

Evapotranspiration is the total moisture loss due to evaporation from the soil, and transpiration plus evaporation from plants. Various methods have been designed to estimate evapotranspiration utilizing records of other weather elements. One of the simplest and probably the most widely used is the C. W. Thornthwaite method that makes use of mean temperature and day-length (9). The N. L. Penman method utilizes more complete weather records and is restricted to areas where measurements of wind, humidity, and sunshine are available (10).

Effect of Evaporation and Evapotranspiration on Crops.—Computations of evapotranspiration may be used as an indication of moisture needs for maximum plant growth and thus serve as a basis for an irrigation program. The computed moisture requirements are more applicable when total plant growth is measured. They are less applicable when the fruit of a plant is to be harvested, especially if a balance between plant growth and fruiting is necessary for maximum fruit set and maturation. In most of the humid areas, the driest weather in the growing season is usually the hottest weather. Because of this and other interrelationships of weather elements, the consideration of rainfall fluctuations may give results similar to the more detailed formula methods for computing moisture deficits. Regardless of the method of computing moisture deficits and irrigation needs, there is no complete substitute for an intimate knowledge of the plant's response to a moisture deficiency.

MICROCLIMATE AND ITS EFFECT ON CROPS

The climate, as discussed thus far, has been the macroclimate. Plants, insects, and diseases live in a climate somewhat different than that measured by an instrument 5 ft above the ground, and their life functions must be studied in relation to their true, or microclimate. For example, many insects stay in the shade of plants during the hot part of the day. During the dead of winter, most insects that survive stay in the protection of debris.

To measure the micro-weather for the United States with the frequency that macro-observations are made would be impractical. Thus, temperature, humidity, wind, and other observations are taken at some standard level above the ground and with a minimum of obstruction to sun and air movement. These macro-observations show a pattern when compared with other readings over a wide area that forms the basis of weather forecasting. Frequently, the temperature readings at two regular instrument shelters 500 mi apart, are closer to each other than they are to the temperature 50 in. below either shelter.

R. Geiger (11) sums it up thus:

> "The difference between the climate near the ground and the macroclimate consists essentially in the proximity of the earth's surface. As the lower limit of the atmosphere, this surface plays an important role in meteorology. The heating and cooling of the atmosphere in the course of the day and according to seasons, takes place in general through it as an intermediary. By evaporation from it, water vapor is given to the air—returning to it again as rain and snow. It acts as a brake on the winds which pressure differences initiate. It is therefore no wonder that the ground air layer shows peculiar climatic characteristics."
>
> "But there is something more. While, in the upper air contrasting conditions which occur are immediately equalized; in the air near the ground they may continue to exist almost side by side, for every convective movement which is initiated is tied up by friction on the surface. Horizontal contrasts are added to vertical. Great climatic differences can result within the shortest distances by reason of the kind of soil, its form, the plants growing thereon, variable shading or sunniness, different wind protection, and many other circumstances."

The effect of plants in changing the adjacent climate is very great. Relative humidity has a direct effect on cotton value at harvest time. In the fall of 1959, measurements were made in the Mississippi Delta of the microclimate in a cotton field that was heavy leafed and in one with very little leaf cover. The relative humidity averaged almost 9% less in the defoliated field during the driest part of the afternoon (12). Evaporation in the heavy leafed field was only half as much as the defoliated field.

Examples of the difference in micro versus macroclimate are endless. The important thing to realize is that there is a difference and then determine what the difference is for each case under consideration.

Although irrigation is the most common method of changing the microclimate there are other practical methods. Mulching the ground around crops for moisture conservation and soil temperature control has been done in some humid areas. Frost protection is the most spectacular effort in controlling microclimate and varies all the way from flooding the cranberry bogs, burning oils, and blowing the air around with giant fans to burying vegetable crops temporarily with soil.

Shelter belts have been widely used in the more arid regions but also have use in the humid area. The reason for the belt is to reduce air flow in the protected area, thereby decreasing evaporation and transpiration. It also has minor effects on temperature and increasing effective precipitation. The net result of all factors contributes to increased yields of certain crops and a reduction of wind damage.

The principle of the shelter belt is to break large eddies into small ones, thus decreasing the wind speed. A belt with about 50% permeability will do this most efficiently, a belt that is solid merely lifts the air current over the barrier and then returns it to the ground a short distance to the leeward. A belt of 50% permeability will reduce the wind speed by 20% over an area from 2 times its height on the windward side to 15 to 20 times its height on the leeward. A lesser reduction extends even further.

CLIMATE AND SECONDARY EFFECTS OF CROP PRODUCTION

The weather not only affects crops directly, as previously stated, but affects production indirectly by influencing crop diseases, crop insects, and many production practices.

Diseases.—According to P. R. Miller, "Given a susceptible plant, the area of occurrence of a plant disease depends primarily on climate" (13).

Plant diseases are caused by fungi, bacteria, and viruses. They are soil-borne, air-borne, and insect-borne. All of these are affected by different combinations of weather.

Many diseases need a good deal of moisture and thus favor the humid region. Cotton anthracnose stays east of a line in east Texas and Oklahoma that corresponds to the 10-in. average summer rainfall. It is a constant and important disease in the humid eastern cotton belt, but is almost non-existant in drier western sections. Apple scab is favored by a cool wet spring and is a humid area disease.

Several important soil-borne organisms cannot stand low temperatures for long. Some of these are Granville wilt of tobacco, southern blight and the Texas root rot of cotton. All attack many kinds of plants and are practically confined to the South. Potato late blight is favored by cool wet weather, however it is not restricted to the North because southern potatoes are grown in the cool moist winter and spring months.

The spread of a plant disease follows a cycle. The plant is infected, the disease grows within the plant tissues, it reproduces and reinfects the plant, and finally it goes into a carry-over stage. All of these stages are affected by weather. The overwintering stage often subjects the disease to its greatest exposure, however, by moving into the soil or debris, or by conversion to a more hardy form, the disease avoids the lethal weather. Certain rusts of wheat overwinter in the South and must then find their way back the next year in spore form by riding the high wind currents. Weather helps spread many diseases by wind and rain, and some soil-borne fungi are spread upward on a plant by the splashing of raindrops.

The weather disease relationship is a complex one and is frequently complicated further by the effect of weather on the disease host. In general, the climate of the humid region favors the developments of many crop diseases.

Insects.—The level of production of many crops is often determined by insects. Weather, directly or indirectly, is the greatest single factor that affects insect populations. It influences the type of insects that live in an area, the numbers, and the seasonal occurrence of the different stages of the life cycle.

The very closeness of the weather-insect relationship is largely due to the fact that insects are "cold-blooded" and respond directly to temperature changes. Most insects thrive in the relatively narrow temperature range that approximates the range of human comfort. Few insects are active above 120° F or below 40°F. Many insects are killed by freezing weather while others survive winter freezes only in a particular stage of their life cycle. For example, the gypsy moth survives the winter as an egg, the brown tail moth as a larva, the tomato hornworm as a pupa, and the chinch bug as an adult. The temperature effect ranges from lethal to optimum with an intermediate zone in which many insects are reduced to a dormant stage.

The time required for various stages of the life cycle of many insects is a direct function of temperature. Within the temperature range of greatest insect activity, there is usually a shorter time necessary for completion of a life cycle. For example, Isley in Arkansas found that the boll weevil developed from egg to adult at 88°F in half the time required at 70°F.

Some insects rely on wind to avoid the winter attrition. The cotton leaf worm, a tropical insect, winters almost entirely south of the United States. As the growing season develops, it moves farther north with each new generation. In the fall, adults appear in the northern states and even in Canada. Winter destroys all stages of this insect in the North and its reappearance is dependent on its spring migration. Corn earworms usually cannot survive northern winters, so they pass the cold season in the South and move northward as moths, later to develop a new generation of larvae that feed on northern tomatoes and corn.

In addition to relying on wind for survival, many insects are dependent on wind and air currents for their spread. Great numbers of insects travel through the air and some move great distances. They are generally concentrated in the lower levels, however, the cotton aphid has been trapped at 13,000 ft and a number of very small insects at 15,000 ft. In sunny warm weather, the air near the ground is subjected to intense heat in the middle of the day. This causes strong updrafts to occur and insects are carried high into the air. After they attain a certain altitude, they are carried along by the prevailing upper winds. Wind direction varies, however, in the eastern humid area, winds from the south or southwest usually accompany conditions favorable for updrafts.

Moisture affects insects indirectly by promoting succulent plant growth, thus providing an abundant supply of food. This is perhaps the main reason insects thrive in the humid area. However, it is true that most insects need a minimum amount of moisture to avoid desiccation, and some insects are dependent on rain puddles for reproduction. On the other hand, very heavy rain is often harmful to insects. It sometimes washes them off the plants on which they are feeding, or drowns some that are in the more sensitive life stages. Some forms of grasshoppers are limited by humid weather which favors the growth of destructive fungi that feed upon them.

Cotton, due to its long field life, has its full share of insects. The following are some of the typical weather insect relations. Cotton boll weevil is limited by low winter temperatures and hot dry summers. During winter, a series of freezes and thaws is more destructive to the weevil and many other types of insects than is extremely cold weather. During the warm thaw period, some of the insects leave their shelter and if a freeze comes with enough suddenness, many are destroyed. Cotton aphids are favored by cool damp weather. The cotton fleahopper is favored by rain and will continue breeding on cotton as long as leaves are succulent. Spider mites, another cotton belt pest thrives in hot, dry weather while a heavy rain often checks an outbreak.

All insects are greatly influenced by weather and all have certain optimum degrees of temperature and moisture, beyond that, generalization is impossible except to note that a great many insects find the humid area the best home. Unfortunately, as with diseases, many insects have optimum weather conditions somewhat similar to the crops upon which they feed and this makes scientific control essential for agricultural efficiency.

Weather for Spraying and Dusting.—Application of dusts and sprays is a necessary part of scientific agriculture in the humid area. These materials may be applied for insect control, disease control, weed control, or as a pre-harvest measure. Whatever their purpose, weather influences their application and their efficiency. Moisture, wind, and temperature singly, or in combination, will largely determine the effect of dust and spray treatments. The choice between a dust and a spray may be dictated by weather conditions.

During the day, the sun warms the soil faster than it does the air just above the ground level. This causes convection currents. These rising columns of warm air are most frequent and reach their highest velocities just after noon on a sunny day. At this time, air tends to be stirred the greatest and a dust or spray treatment has the poorest chance of reaching its target. Sprays provide considerably more leeway in this respect than dusts. Air movement is usually least in early morning hours and late afternoon hours. Exceptions occur when a front or a strong pressure system moves through an area.

Wind velocity is a key factor in determining whether a dust or spray may be applied satisfactorily. Wind speeds in excess of 8 mph to 10 mph will cause considerable drift of dusts, and to a lesser degree, sprays. The drifting materials will be less efficient to the crops intended for treatment and may even produce harmful effects on adjacent crops. This is especially true with some herbicidal materials in which drifts have caused damage to crops more than a mile from the treated areas.

In addition to requiring calm weather conditions, most of the dusts also require dew to be effective. The application of harvest-aid dusts is a notable example. Where these materials are to be applied, the crop producers have come to depend on the dew forecast in making plans for treatments.

Wind and dew influence the application of sprays and dusts, but in most cases, rain determines the degree and length of effect of treatments. Rain immediately after application of dusts or spray will reduce the treatment's effect, and depending on the amount of rainfall, may render the treatment completely ineffective.

Temperature is an important consideration in selection of chemicals to be used as dusts or sprays. In insect control, some chemicals break down rapidly at high temperatures, and are effective for only very short periods, whereas others are effective only at high temperatures. Low temperatures generally reduce the activity of harvest-aid chemicals. Since some chemicals can tolerate lower temperature than others, the selection of the material will be influenced by prevailing and anticipated temperatures.

Tillage and Weed Control.—The threefold purpose of soil tillage is to improve soil structure, prepare a seed-bed, and to control weeds. Humid conditions make it difficult to achieve the second and third objectives without having adverse effects on soil structure. Tillage of the soil when the moisture content is high may cause soil compaction. Excessive tillage of rolling land, especially when no immediate cover is to be provided, will promote serious erosion.

In recent years, the use of chemicals for weed control has expanded rapidly. This practice is even more dependent on climate than mechanical methods of weed control. Moisture is required to activate the chemicals, but heavy rains may cause them to be lost. Low temperatures may prevent the chemicals from having any effect on weeds, while high temperatures may cause the effect to be short-lived.

In general, weather conditions that favor crop production, encourage weed growth and, at the same time, makes control measures difficult.

Fertilization.—Crop fertilization is necessary in most humid areas for economic yields. For many years fertilization was practiced only in the humid areas and, as late as 1950, 95% of the commercial fertilizer was used in the eastern half of the nation. Without considering indirect relations between fertilization and climate, there are several in which climate exerts a direct influence on crop fertilization programs. Without elaboration a few of these relations follow:

1. Moisture is necessary for the plant utilization of commercial fertilizers.
2. High rainfall and high temperatures promote rapid decomposition of organic matter. Thus, in the warmer portion of the humid area, climatic conditions prevent a build-up of organic matter and soil nitrogen. The rate of activity of soil micro-organisms above 45°F increases two to three times for each 18° rise in temperature if moisture is adequate and the soil is not highly acid.
3. Rainfall causes soil nutrients to be leached from the soil. The greater the rainfall and the coarser the soil texture, the greater will be the nutrient loss.
4. Loss of soil bases by leaching due to heavy rains increases soil acidity. This may result in a need for a liming program.
5. Extended periods of wet weather with high temperatures on fine textured soils may cause loss of nitrogen by denitrification.

Because of the temperature difference within the humid area, two greatly different fertilizer programs may be applicable. In the areas with lower temperatures, it may be practical to fertilize the soil. In the warmer areas, the crops should be fertilized rather than the soil.

WEATHER RELATIONS OF CERTAIN CROPS

Cotton.—Three climatic factors localize the area of cotton production: a long frost-free growing season, a large supply of moisture, and a plentiful amount of sunshine. Plant variety and management practices have succeeded in modifying the requirements slightly, however, cotton requires a frost-free season of about 180 to 220 days, an optimum supply of 20 to 28 in. during the growing season, and sunshine during over half of the daylight growing period.

Soil temperature should reach an average of 70° for optimum seedling emergence although germination requirements are slightly less, and a practical average for planting is about 65°. Soil moisture must be adequate to germinate the seed, or showers must occur in a week or so after seeding. Frost occasionally kills the plants and necessitates replanting especially in the northern part of the cotton belt, however, cool wet weather that promotes seedling diseases is more often the limiting factor.

Cotton's very long period in the field subjects it to all types of weather damage. Winds and hail do physical damage, very heavy rains sometimes cause flooding, and the weather is usually favorable to many insects and diseases. Cotton fruits over a long period of time. Most crops are especially sensitive to moisture deficiencies and excesses during the fruiting period. So it is with cotton, and this spreads the sensitive time over a period of months, not weeks or days.

During the summer, a well distributed rainfall is desirable. Frequent changes from cool-and-damp to hot-and-dry causes a shedding or dropping of bolls. A deficit of sunshine also causes shedding, as do a number of other adverse weather conditions.

During the harvest, cotton continues sensitive to weather. Temperatures of freezing or much below cause bolls to rot. A light frost causes the leaves to drop and is beneficial to the harvest. Chemical defoliation is common in humid regions. It disposes of the leaves and mechanical pickers gather a smaller percentage of trash. When cotton is picked wetter than about 10% moisture content, the various quality measures suffer. Measurements made in defoliated cotton show that "safe" picking time is increased about a hour, due to the drier microclimate that prevails in daytime in humid areas.

Corn.—Because of its adaptability, corn is grown successfully in every state of the country and at altitudes from sea level to 10,000 ft, however, it is concentrated in the middle of the country, in the corn belt. Corn is a warm weather crop, and the corn belt with a mean summer temperature of 70° to 80° and a frost-free period of 140 days suit it well. Warm nighttime temperatures are important, and in this region the average summer minimum is over 58°.

The minimum temperature for germination of most corn varieties is 50°, and very little growth is made after the plant is up when the temperature is below that level. Prolonged temperatures of below 45° will kill many varieties of corn. Most plants will stand a light freeze in the seedling stage, but after that, a freeze will kill all but the most hardy plants.

Corn flowers and ripens much sooner when grown at 80° than at 70°, and temperatures as low as 60° greatly retard maturity. Extremely hot weather may injure the plant, especially when combined with deficient moisture. The time of tasseling is a most sensitive period.

The best corn regions have an annual rainfall of 25 in. to 40 in., except for areas of irrigation. The moisture required during the growing season for optimum production varies from 20 in. to 28 in. depending on soil type, variety, temperature, and evaporation conditions. The critical period is during the three weeks following the initial show of tassels.

Extended periods of cloudiness harm corn, which requires abundant sunshine. At the time of harvest, sunshine and open weather are needed to bring the moisture content down to a safe level for storage. Corn is usually picked with a moisture content of 13% to 30% moisture. To be safe in storage bins, it should contain less than 15% moisture. When the first freeze catches corn in an immature state, and with a high moisture content, "soft" corn results. This is unsuitable for processing, and the best use of soft corn is prompt feeding to avoid spoilage.

Corn is a short-day plant, flowering is speeded and vegetative growth slowed by short days. Varieties that have been developed for a particular area suffer by the change of day-length when they are moved either north or south.

Thunderstorms, wind and hail storms have a relative high frequency in the corn belt, and at times cause considerable damage. The mechanical picker emphasizes the problem of broken stalks such that the ears cannot be reached. The European corn borer often attacks stalks, weakening them and making them further susceptible to wind damage.

Sorghum.—Weather requirements as well as uses of sorghum are similar to corn with some major differences. Sorghum seems to tolerate extreme heat and drought better than most crops, however, extremely high temperatures during the fruiting period does reduce seed yield. The most favorable mean temperature is about 80°F. The minimum temperature for growth is about 60°F. It is a short-day plant.

Sorghum does well with limited rainfall, but is highly productive in humid and irrigated areas. The plant has more secondary roots than corn and smaller leaf area. Also, the leaves and stalk wilt and dry more slowly than those of corn; a waxy cuticle seems to slow drying. Sorghum plants remain dormant during drought, but begin again after sufficient rain. Its resistance to heat, drought, and certain insects make it adaptable in some areas where corn is not. A number of economic qualities make it secondary to corn in most areas. In the Mississippi Delta, land efficiency is being increased by harvesting a double crop; one in the spring or early summer with a head chopper, and another head crop or the entire plant again in fall. This double cropping is especially desirable in high-value land.

Small Grains.—Wheat production is not as extensive in the humid regions as in the dry plains states. Excessive moisture, especially when combined with high temperatures, favors the diseases of wheat. Heavy rains cause lodging or breaking of supporting straw. General wet weather at harvest time is quite detrimental. Soft red winter wheat varieties are general in the humid wheat states. These are planted in the fall and must pass through the winter cold.

The Hessian fly is a serious pest of wheat, particularly in the humid East. To minimize damage by this pest, the fall planting date is established late enough that the main brood of flies will have emerged and died before the young plants appear above the ground. The "fly-free dates of planting" are well established in the humid region and are well adhered to. A number of other insects and diseases also attack wheat in humid areas.

Most oats are best adapted to cool regions with an annual moisture supply of 30 in. or more. Hot, dry weather is detrimental during most of the life of oats, although some varieties are more tolerant than others. At time of heading and ripening, hot, dry weather causes premature ripening and the grain is poorly filled. Lodging is a serious problem in most sections and wind can do considerable damage to oats.

Rice requires considerable heat and moisture. A mean temperature of about 70°F is necessary for the long growing season of 4 to 6 months. A constant supply of fresh water for irrigation must be available. In the South, rice is usually submerged from the time the seedlings are 6 in. to 10 in. high until just before the harvest. The land is flooded to a depth of 1 in. to 2 in. at first, and gradually increased to 4 in. to 6 in. through the summer. The water requirement ranges from 24 in. to 48 in. The time of submergence averages 3 to 5 months in the South while in California the time of submergence is a little longer. Even for rice, however, a period of open weather at harvest time is necessary. If the open weather is accompanied by an abundance of sun and hot, drying winds, the grain is subject to cracking.

Legumes.—Soybeans have climatic requirements quite similar to corn, and in fact, the corn belt is the most concentrated soybean area. Germination is the most sensitive period in soybean growing; either drought or weather that is too wet is harmful. Soybeans are somewhat less susceptible to frost injury than is corn, and light frosts are not injurious to either young or mature soybeans. The plant will survive a short period of drought after it is established, but a water shortage brought on by the combination of hot and dry weather is one of soybeans' worst threats. High temperatures alone will lower yield and quality of oil. The soybean is a short-day plant and thus matures in the shorter days in fall, and in the South, can be harvested in time to plant a fall oat crop.

Alfalfa, while adapted to widely different climatic conditions, makes its best growth in the sub-humid areas, but where irrigation is available. In areas of irrigation, good drainage is essential. It can stand extremes of heat and cold. It can stand long periods of drought, but it grows very little and merely sustains its life. The growing of alfalfa has increased in the last 10 yr to 20 yr (since about 1940) because of increased knowledge of fertilizer requirements.

Sweetclover is also adapted to a wide range of climates, and is grown in every state although its greatest concentration is in the great plains, and the north-central states. Sweetclover is grown in areas where spring and summer rainfall is 15 in. to 20 in. or more. It survives drought similar to alfalfa, however, moisture and cool temperatures are necessary for germination and early seedling growth.

Red clover is widely grown in the cooler humid parts of the country for soil improvement and forage. Rainfall appears to be the main limiting climatic factor in the northern part of the humid area, however, the cold temperatures of winter occasionally kill a stand. High summer temperatures do not injure the plant except in connection with a water shortage. Alsike clover requires a cool climate with ample moisture; it withstands severe winters better than red clover. White clover grows best in moist, cool weather and will withstand extremes in temperature better than either red clover or Alsike clover. White clover is grown in the southeast as a winter annual and dies in the early summer after seed is produced. Crimson clover has similar climatic requirements and is also widely grown in the South as a winter annual.

Tobacco.—Tobacco seed is usually planted in seedbeds that are protected by glass or cloth covers. Germination and growth are slow with mean temperatures of 50° to 60° and the optimum is about 75° to 80° whereas 95° or higher is very detrimental. In the northern states, the young plants are transplanted in late May or early June, and it is rare that they are damaged by late freezes. Delayed planting there involves the risk of fall freezes, and at the time of maturity, the crop is quite susceptible to damage. In the South, the crop is transplanted early because high soil temperatures and strong sunlight can permanently stunt or kill the plants.

A wet winter and spring may make seedbed preparation difficult and interfere with the process of soil sterilization. The amount of rain and the condition of the soil are quite important at the time of transplanting. Very dry weather entails the artificial watering of each plant as it is set. Excessive rain and humidity are conducive to damping-off and other diseases. Temperatures above 95° and bright sunshine may burn the leaves, especially during periods of drought.

The large leaf area makes for high transpiration and tobacco needs plenty of rain. During the 90 days of the normal growing season, it receives about 11 in. to 14 in. in the northern and central areas, and 15 in. to 16 in. in the extreme southeast. The large leaf area also makes the plant very sensitive to wind and hail storms.

The modification of microclimate is practiced during the curing time as well as during seedling. Humidity control is accomplished by different ventilation methods in the barn. The several types of tobacco have their own optimum weather condition. Shade grown tobacco, growing the crop under nets, in Connecticut and other areas, is an example of the modification of microclimate to produce a desired quality of tobacco, fine cigar wrappers.

Sugar Crops.—The two sugar crops, beets and cane are favored by quite different climatic optimums. The former is grown in the northern part of the humid area, and the latter in the semi-tropical areas of Louisiana and Florida.

Sugar beet seed will germinate at a temperature of only slightly above freezing, but to reduce rot, should not be planted in temperatures less than 50°, and will germinate and grow even better with a temperature near 60°. The seedlings are quite sensitive to cold and will be killed by temperatures in the high 20's, but after a week or so, they are able to withstand frost and freezing temperatures. During the summer, crop growth is efficient with a mean temperature of 70° or a little higher. The accumulation of sugar is retarded by temperatures above the mid 80's which, in part, accounts for its ill adaptation in the South. Adequate moisture is necessary in the summer. The seasonal rainfall for efficient production is about 10 in. to 15 in., and areas with less than this amount are normally irrigated. Cool fall temperatures in the north of the humid area promote increased sugar content.

Sugar cane is commercially planted in stalk strips 2 ft to 3 ft long. In Louisiana, it is normally planted in the fall, and only a few areas a little further north are planted in spring. The plant requires 8 to 24 months to reach maturity. Each crop is usually cut twice. Following 8 months of temperatures high enough to promote vegetative growth, a short cool period promotes sugar accumulation. During the winter, the stub from the first year's growth, or the new stalk seedlings are covered with dirt as a protection from the cold. In spring, the dirt is removed leaving a narrow bed that warms up quickly to start spring growth. In addition to needing relatively high summer temperatures, as in opposition to sugar beets, the cane needs a bountiful supply of moisture, however, adequate drainage for excess rain is also essential.

To a degree, sugar production in the humid areas is favored by cool weather in the North for beets, and warm weather in the South for cane.

Vegetables.—Vegetables need a good deal of rain and are chiefly grown in areas of 30 in. to 40 in. of yearly rain or areas with irrigation. Of all the vegetables, only a few varieties of beans can be considered dry-weather plants. Low humidities with drying winds of summer damage the flowers of certain plants and retard the number of fruit that can be set.

Temperature has less influence on vegetable production than moisture, as nearly all sections have weather warm enough for all except a few tropical vegetables. The season of warm weather, of course, limits winter production to the extreme South. In the mid-south, some areas grow a double crop, one in spring and another in fall, avoiding the extremes of winter and summer.

In the South, the high summer temperatures, even when combined with adequate moisture limits vegetable production. Insects and diseases thrive in the warm moist summer weather. The vegetables develop rapidly and must be harvested at the proper moment to avoid over ripening. This ripening goes on even after the crop is harvested. In some plants, the high temperatures and consequent high rate of transpiration accelerate the loss of sugars and thus changes the flavor, even when there is no visible change. Lettuce is subject to leafburn with high temperatures, and other leafy vegetables, such as spinach and cabbage, lose their best flavor. Even some warm weather crops suffer from high temperatures. Tomatoes scald when exposed to the sun and even hardy beans may be damaged. For these reasons, most of the summer vegetables are grown under the cooler conditions of the North.

The potato is a cool weather crop and grows best in the northern part of the humid region, however, a good early variety crop is grown in the South. High temperatures injure the tuber. Freeze injury is most common in the South and affects the winter or early spring crop. In the North, the season is cut short at times by early freezes, however, if the freeze is not too severe, the harvest is aided because the plants drop their leaves. Potatoes that are frozen are seriously damaged. Even when stored at a temperature just above freezing, they are rendered unpalatable as the starch is converted to sugar. The soil moisture must give an even supply of water or the potato will be knobby and lowered in the market value. Potato late blight is a serious problem, because the weather that favors this disease, also generally favors potato growth. There are a number of forecast procedures to warn of disease development. All are dependent on optimum temperatures, 50° to 80°, and wet leaf surfaces. The Department of Agriculture in the northcentral states provides forecasts of this disease's development, utilizing field reports and long-term forecasts of the USWB (14).

Sweet potatoes will not thrive in cool weather. No other common crop in the United States will stand so much heat and very few require as much. Most of the commercial crop is grown south of the 75° average summer temperature line, except for the area of Maryland, Delaware, and New Jersey, and some smaller areas of Iowa through southern Indiana. The sweet potato is a source of starch, and for this use, the crop is limited to the South with a

July-August temperature above 80°. The western limit of culture is in central Texas or just beyond the humid area. The crop does best where annual rainfall is over 35 in. except for irrigated areas. Day-length does not affect root development, but the vegetative growth is favored by long summer days. Sweet potato roots are bedded in warm soil for 4 wk to 6 wk, and then the sprouts are pulled and planted after the mean temperature approaches 70°. Prolonged exposure of leaves or roots much below 50° will damage or destroy the crop. Only a very light frost will kill the foliage.

Fruits.—Tropical fruits, such as bananas, breadfruit, coconuts, Brazil nuts, coffee, and cocoa cannot withstand a freeze, and even periods in the low 40's hurt the crop. This limits commercial production to only the extreme southern part of the eastern humid region.

Sub-tropical fruits, such as citrus, olives, figs, and dates, will tolerate temperatures slightly below freezing, if in the correct stage of development. They need a little cooler weather than is found in the tropics for proper development.

Most citrus have a optimum temperature from the mid 70's to the low 90's. Growth practically stops below 55°F or above 100°F. Normal yearly precipitation requirements are estimated at about 35 in. Winds not only give some direct injury, but loss of fruit occurs due to excessive drying. Fairly high relative humidity seems to be associated with high quality fruit.

Date palms can endure lower temperatures than citrus and in a dormant state, are rarely injured by 20°F. Figs can stand 15°F in a dormant state. Moisture requirements of dates and figs are less than those of citrus, and wet weather is injurious by direct damage and by promoting insects and diseases.

Hardy fruits, such as apples, peaches, pears, cherries, plums, prunes, grapes, and apricots can stand temperatures considerably below freezing during the dormant stage. Deciduous fruit trees all require a certain amount of cold during the winter or the buds will not open in spring. Apples and most peaches need 600 to 900 hr of temperatures below 45°F. This limits their production along the Gulf coast.

During winter, root injury due to very cold weather can kill trees, but a cover crop protects that area. Fruit buds are sensitive to cold and are frequently killed by low winter temperatures that do not injure the rest of the tree. During the spring, a sharp freeze following a mild period can kill apple trees with readings of zero in the North and 15 in the South. Various fruits differ greatly in the amount of heat needed to expose the sensitive flower parts, and this determines the susceptibility to spring frosts. The apple requires the greatest amount of warm weather to bloom, and therefore is the least likely to be caught by spring freezes. The apricot requires the least heat and is the most likely to be injured.

All fruits in this group need ample water and 30 in. of yearly precipitation in the natural limit. The early ripening fruits, such as cherries and apricots, do not need quite as much water as the late maturing peaches, apples, and pears. Cherries thrive with an average summer temperature of about 65°F, apples 65° to 75°F, pears a little higher, and peaches in areas of above 75°F. Most fruits are susceptible to disease and many have the same optimum conditions, thus good fruit weather is usually good disease weather.

Grapes that grow in the humid area will endure temperatures that kill peach trees and they approach the hardiness of apples. All grapes are resistant to drought as compared with tree fruits. Many types of grapes are susceptible to fungus diseases, however some varieties grown in the southeast have resistance. For quality, grapes need a good deal of sunshine. In some European areas, sunshine is increased by the use of reflectors.

Strawberries are grown in all parts of the humid region. In the North, they must be protected by mulching or some other process from the cold, and in the humid East, varieties that are resistant to fungus diseases must be used. Most strawberry varieties are short-day plants. In the North, all fruit buds are formed in the fall; in the South, fruit buds form in the fall and also in the spring.

Pecans need a 200-day frost-free season and a long hot growing season to mature the nut. A shorter period of cold weather is sufficient to break dormancy in pecans than most deciduous fruits. About 40 in. to 50 in. of well distributed rain or irrigation is normal for a mature orchard.

Walnuts are quite susceptible to fungus diseases in very humid regions. Very high summer temperatures are detrimental to walnuts, and in winter, cold requirements to break dormancy are high in many varieties. Growth starts early in spring and late frosts are a serious hazard. The black, or American walnut is a native of the United States and grows well in all but the extreme north and south areas of the humid region.

SUMMARY

Climatic influence on crop production goes far beyond the obvious limiting effect of drought or flood and searing heat or freezing cold. Each crop has its own optimum value of the weather variables: precipitation, temperature, sunlight and day-length, humidity, and evaporation. Also, the various farm practices necessary to raise a crop efficiently are greatly influenced by these weather variables. In addition, the weather variables themselves have great variation over the humid area. To evaluate an agricultural procedure, the variation of weather, the crop-weather relationship, and the crop practice-weather relationship must all be considered.

Specific considerations for the engineer engaged in designing procedures for increasing farm efficiency in humid areas include the following: (1) The indications are strong that supplemental irrigation during certain critical periods of the crop cycle will increase most crop yields, even in humid regions. (2) Only a small part of the heat received from the sun is actually put to efficient use, and this large potential power supply might be utilized by modified farm practices. (3) The large moisture supply makes the control of insects, diseases, and weeds, and the techniques of tillage and fertilization of particular importance in humid areas. (4) The modification of certain phases of the microclimate, temperature, sunlight, and evaporation, has demonstrated spectacular increases in crop efficiency in small areas and awaits the development of procedures for the spread of these modifications over large areas.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3226

FUNDAMENTAL CONSIDERATIONS IN HIGH-RATE DIGESTION

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With Discussion by Messrs. F. Sulzer; C. E. Keefer; M. T. Garrett, Jr.; and Clair N. Sawyer and Jay S. Grumbling

SYNOPSIS

A comparison of the results obtained in high-rate digestion using shortened digestion times versus sludge concentration. The importance of fixed solids loadings in relation to mixing problems is described. Concepts of unimolecular and first order reaction kinetics are developed for anaerobic digestion.

High-rate digestion of sewage sludge has been studied extensively in laboratory, pilot, and plant scale operations by several investigators.[3,4,5,6,7,8,9,10,11] The senior author has summarized the status of knowledge as it existed in published form in 1957[12] and it is not the purpose of this paper to review

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Note.—Published essentially as printed here, in March, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2411. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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[3] Sewage and Industrial Wastes, by J. H. Blodgett, Vol. 28, No. 926, 1956.

[4] Wastes Engineering, by J. H. Blodgett, Vol. 24, No. 608, 1953.

[5] Ibid., by P. F. Morgan, Vol. 26, No. 462, 1954.

[6] Sewage and Industrial Wastes by H. K. Roy, and C. N. Sawyer, Vol. 27, No. 1356, 1955.

[7] Journal, Boston Soc. of Civ. Engrs., by H. Schmidt, and C. N. Sawyer, Vol. 42, No. 1, 1955.

[8] Sewage and Industrial Wastes, by K. L. Schulze, Vol. 30, No. 28, 1958.

[9] Ibid., by W. N. Torpey, Vol. 26, No. 479, 1954.

[10] Ibid., by W. N. Torpey, Vol. 27, No. 121, 1955.

[11] Ibid., by H. R. Zablastzky, M. S. Cornish and J. K. Adams, Vol. 28, No. 1299, 1956.

[12] Biological Treatment of Sewage and Industrial Wastes. by C. N. Sawyer, Reinhold Publishing Corp., New York, 1958, Chpts. 1-5.

such material but rather to summarize new information and concepts developed since that time.

The investigations on high-rate digestion have established that there is one fundamental requirement, in addition to the normal ones, for the successful operation of high-rate units. This requirement is that the contents of the digestion tanks be thoroughly mixed on a more or less continuous basis.

Mixing accomplishes three major objectives all of which contribute toward keeping the biological forces operating at or near peak capacity at all times. First, the active organisms are kept continuously in contact with the food supply; second, the food supply is uniformly distributed and made as available to the organisms as possible; and third, the concentration of inhibitory biological intermediates and endproducts are maintained at minimum levels. All of these factors serve as a means of keeping the working population of organisms performing at peak efficiency.

From the engineering viewpoint it is not enough to just have the microbial population living in an ideal environment. Unless this advantage can be harnessed to do more than the normal amount of useful work, little has been gained. There is nothing the engineer can do, beyond maintaining an optimum environment that will change the rate at which individual organisms will work for him, other than to apply the well-established principles which govern first-order reactions. These are the principles that are applied, knowingly or unknowingly, when reduced detention times are employed to increase the organic loading on digestion units to as much as three or four times conventional. The same principles have been applied to the operation of high-rate trickling filters and the high-rate activated sludge process.

Sludge differs from sewage in several respects, but its one unique characteristic is the relative ease with which sludge can be concentrated under the influence of gravity. This property allows the engineer to increase organic loadings markedly without changing the detention time. Organic loadings can usually be increased by a factor of 1.5 to 2 by this practice alone, and usually result in improved operation of digestion units.

The ultimate in organic loadings is achieved by using a combination of the two systems mentioned. Fig. 1 illustrates the relative organic loadings that can be expected from each of the systems individually and in combination. The practicability of handling organic loadings in excess of 500 lb per 1,000 cu ft per day has not been demonstrated.

KINETICS OF SLUDGE DIGESTION

The degradation of organic matter in biological systems, aerobic or anaerobic, is brought about by micro-organisms, mainly bacteria, and the reaction may be represented for anaerobic systems as follows:

$$\text{Organic Matter} + \text{Bacteria} \rightarrow CH_4 + CO_2$$

Because two reactants are involved the reaction is binary in character but, since the bacteria are not consumed in the reaction, the reaction is actually first order in character, and the kinetics are the same as those for unimolecular reactions, as modified by the diverse nature of the organic matter and changes in bacterial populations. In a digester, which is being fed more or

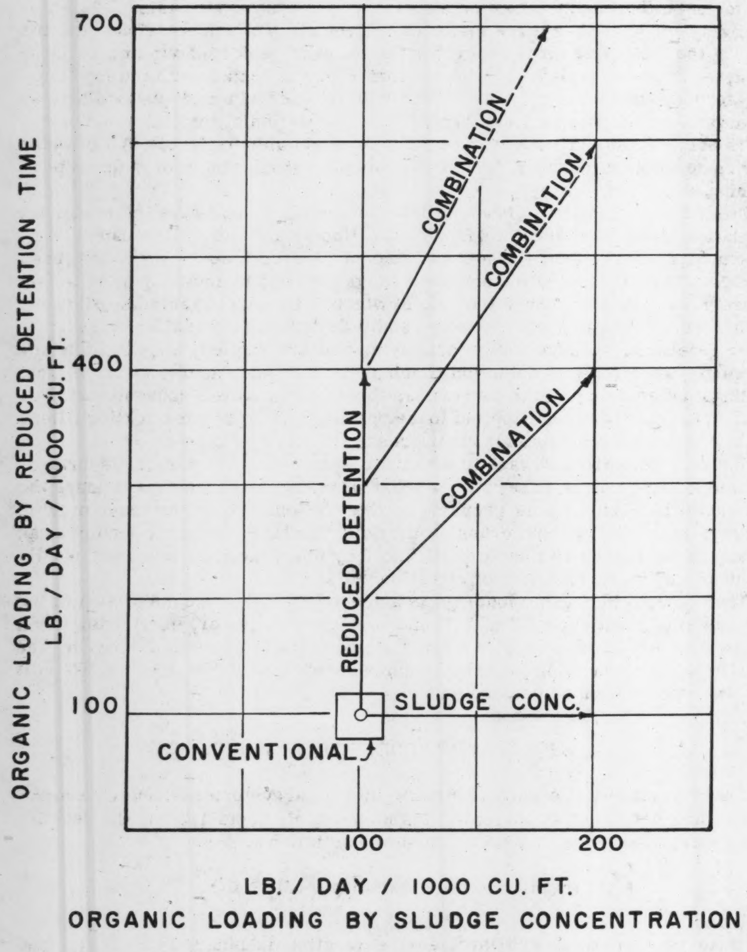

FIG. 1.—INFLUENCE OF SLUDGE CONCENTRATION AND/OR DETENTION TIME ON SOLIDS LOADINGS TO DIGESTION UNITS

less uniformly on a day to day basis, it is reasonable to assume that the working population of bacteria is a constant. This leaves only the organic matter for consideration.

Studies of the kinetics of unimolecular and first order reactions have shown that the rate of such reactions is a function of the concentration of the reacting substance at any given time and may be expressed as follows:

$$\frac{-dC}{dt} \alpha C$$

and

$$\frac{-dC}{dt} = k C$$

This means that the amount of material undergoing change per unit time decreases with time and is analogous to the decay of radioactive materials, a true unimolecular reaction. Fig. 2 shows the mathematical aspects of unimolecular or first order reactions in graphical form. From this curve or its mathematical interpretation, we have the fundamental basis for justifying decreased detention times as a means of accommodating greater organic loadings on digestion units. For example, if a digestion tank is operating at ten or more days detention under conditions represented by the curve in Fig. 2, it is obvious that the detention time could be decreased to seven days without serious effect on the degree of stabilization produced. As detention times are decreased below seven days it is to be expected that rapid deterioration in sludge quality will occur. Failure of the system can be expected long before one or two days detention is reached because of limitations of the biological system to cope with environmental changes such as decreased buffering capacity.

The discussion so far has been based on the assumption that all of the components of sewage sludge are stabilized at the same rate. Actually this is not the case. Sludge consists of a mixture of materials with widely varying biological assimilabilities or, in other words, markedly differend k- values. As a result, the reaction rate is influenced greatly by the easily assimilated high k-value materials during the early stages of digestion, while the materials with low k- values govern the over-all rate in the later stages.

The concepts of unimolecular and first order reaction kinetics also serve as a basis for predicting that the behavior of digestion units fed sludges of different concentrations (within limits) will be essentially the same. For example, if one digester is fed a 4% sludge and another unit is fed an 8% sludge over a prolonged period of time, the population of active organisms in the second digester will be essentially twice as great as in the first. The controlling factor then becomes the concentration of food and, since

$$\frac{-dC}{dt} = k C$$

the amount of organic matter destroyed per unit time in the two digesters will vary in the ratio of 1 to 2. On this basis gas production and formation of other biochemical end products should be in direct proportion to the concentration (%) of solids in the sludges fed, when all other variables are held constant. Another factor of some importance concerns the character of the digested sludge withdrawn from the units. For a given detention time, the digested

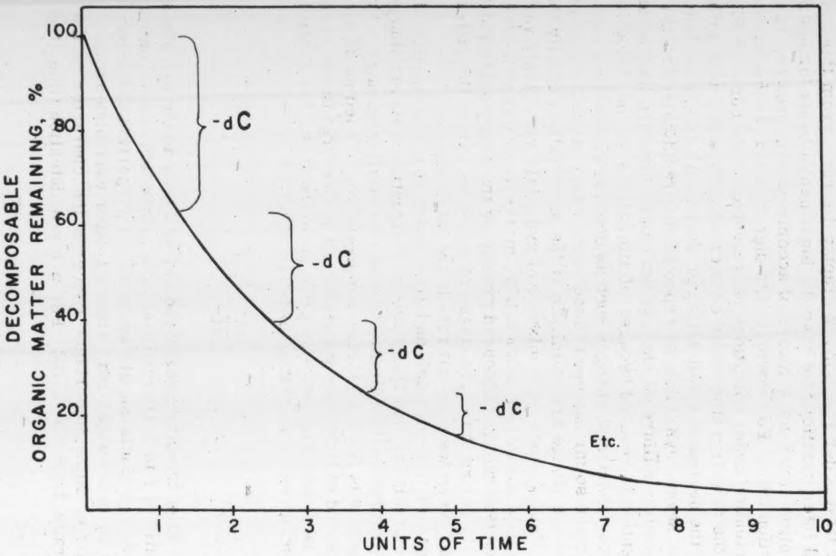

FIG. 2.—ILLUSTRATING THE KINETICS OF FIRST ORDER OF UNIMOLECULAR REACTIONS

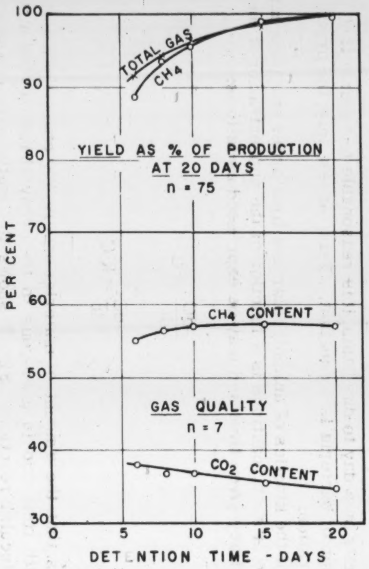

FIG. 3.—EFFECT OF DETENTION TIME ON GAS PRODUCTION AND COMPOSITION (REF. 7)

sludge should be of essentially the same quality regardless of the concentration of solids in the sludge fed.

EXPERIMENTAL VERIFICATION OF THEORETICAL ASPECTS

Detention Time.—An extensive series of investigations were conducted by H. K. Roy[6] to determine the effects of detention times varying from 6 to 20 days on gas yields, composition of gas produced, volatile matter destruction, and quality of digested sludge. The results of his studies on gas yield and gas composition are shown in Fig. 3. From it, it may be noted that total production of gas and of methane decreased gradually as the detention time diminished. At six days detention the yield of methane and total gas were 88.8% and 91.6%, respectively, of the amount produced at 20 days detention. The trend of the curves from the tenth to the sixth day is increasingly abrupt and indicates that process failure could easily occur at detention periods less than six days.

The composition of the digested sludges withdrawn from the units is shown in Table 1. These data correlate well with the gas production values.

Of particular note are the volatile solids and grease values. When these values are converted to a percent destruction basis, the values given in Fig. 4 are obtained. From these data, we note that there is a marked decrease in grease destruction as detention times fall below 20 days, and the trend appears to be approaching a critical stage at a 6-day detention time. This is, undoubtedly, the major reason for the rapid decline in methane yield shown in Fig. 3. The trend of the volatile matter destruction curve appears to be much less critical but this is probably misleading. Actually, the volatile matter determination does not distinguish between biologically assimilable and nonassimilable substances. Therefore, interpretations from the curve should take this into account.

TABLE 1.—COMPOSITION OF WITHDRAWN DIGESTED SLUDGE[a]

Determination	Digester Detention Period, Days				
	6	8	10	15	20
Total solids, %	3.15	3.05	2.99	2.92	2.77
Volatile solids, %	58.3	57.8	56.4	55.6	55.2
Volatile acids, ppm	270	140	110	110	110
Alkalinity, ppm	2,540	2,640	2,720	2,920	2,930
pH	7.0	7.05	7.15	7.30	7.45
Ammonia-N, ppm	400	418	434	464	465
Grease, %	9.50	8.85	8.05	7.15	6.65

[a] Based on 75-day period; average of 11 samples (from Ref. 6).

Sludge Concentration.—The effect of sludge concentrations on the anaerobic digestion process has been investigated on pilot or plant scale by W. N. Torpey,[9,10] F. ASCE and Brisbin.[13] The results of laboratory studies have been reported by H. Schmidt[7] and K. L. Schulze.[8] The data obtained by these investigators, particularly those of Schmidt,[7] have indicated that the degree of digestion obtained for a given detention time was more or less independent of the concentration of sludge fed to the units. However, the results of both

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[13] Proceedings, Seminar on Waste Water Treatment and Disposal, by R. S. Rankin, Boston Soc. Civ. Engrs., 1957.

Schmidt and Schulze suggest that there is a limitation to the concentration of sludge that can be fed and still maintain stable operation. Because of this possibility, and other reasons, investigations were started in our laboratory to study the subject in some detail.

Laboratory equipment similar to that used by Roy,[6] utilizing gas recirculation for mixing, was employed for the studies. The digestion units have been fed once daily with sludges containing 4%, 6%, 8%, and 10% solids on a 15-day detention basis. Gas recirculation for mixing was intermittent (only 15 min of each hour). The sludge used was a mixture of primary and activated sludges obtained from the Leominster, Mass. sewage treatment plant. The sludge was concentrated by the system of flotation employed in the Laboon Process,[14] using a 24 hr or 48 hr contact period. Subnatant liquor from the flotation unit was used to dilute the thickening sludge to the proper concentrations for feeding the various units.

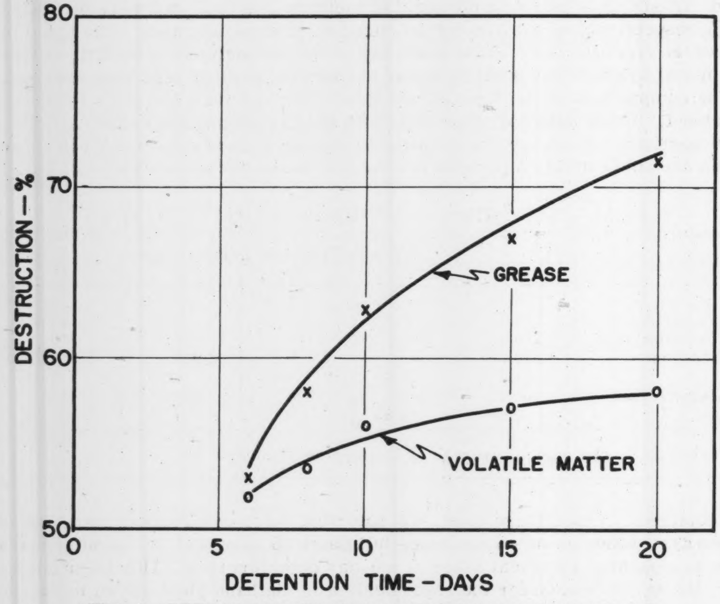

FIG. 4.—EFFECT OF DETENTION TIME ON DESTRUCTION OF ORGANIC MATTER (REF. 7)

The data obtained on gas production and gas composition are shown in Fig. 5. These indicate that there was some deterioration in gas quality at the 10% solids level and that there was a significant decrease in total gas and methane yields as the concentration of solids increased in the sludge fed. The yield of methane for the 10% sludge was only 92% of that produced by the 4% sludge. This illustrates the point that concentration of sludges cannot be accomplished without some liquifaction of the sludge and loss of gas forming materials in subnatant or overflow liquors. It may be argued, however, that concentration by the Laboon Process gives an exaggerated condition because of the high temperatures and prolonged periods involved in the flotation units.

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[14] Sewage and Industrial Wastes, by J. F. Laboon, Vol. 24, No. 423, 1952.

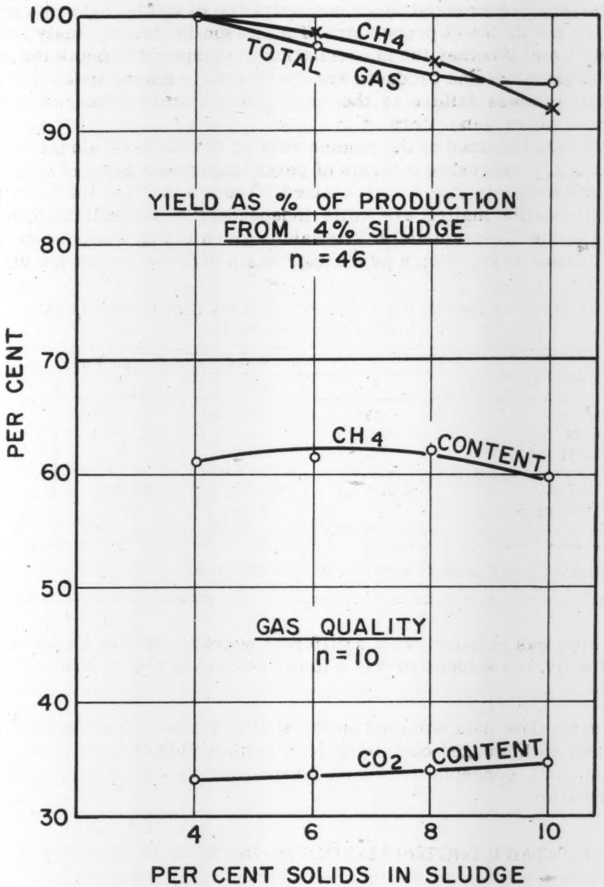

FIG. 5.—EFFECT OF SOLIDS CONCENTRATION IN SLUDGE FED ON GAS PRODUCTION AND COMPOSITION

The data obtained on the digested material, as withdrawn from the digestion units, are shown in Table 2. The volatile solids and grease data show that the extent of destruction of organic matter was essentially the same in all units. The volatile acids and pH data indicate that very favorable conditions existed in all units for proper functioning of the methane forming bacteria. The decreased methane content of the gas, therefore, must be attributed to other reasons. This is most likely due to the accumulation of nongaseous biochemical end products that remain in the digesting mixture. Some of these are indicated by alkalinity and ammonia nitrogen in Table 2. It may be noted that the amounts of these materials increased as the concentration of solids in the sludge fed increased, but not in direct proportion. The reason for this anomaly is not evident at this time. Whether the accumulation of compounds containing ammonia or other biochemical end products are the cause of reduced methane formation and eventual process failure is the major subject of the research, now under way, in the writers' laboratory.

When the data obtained on the studies related to effects of sludge concentration (Table 2) are expressed in terms of percentage destruction of organic matter, the data shown in Fig. 6 are obtained. These show that the destruction of grease and volatile matter are quite independent of the solids content of the sludge fed to the digestion units. The values given in Fig. 6 should not be compared with those in Fig. 4 on a percentage basis because the sludge used in the latest studies was obtained from a different source. It was a mixture of primary and activated sludges having a lower content of grease and volatile matter.

TABLE 2.—COMPOSITION OF WITHDRAWN DIGESTED SLUDGE
(15-Day Detention Period)

Determination	% Solids in Sludge Fed[a]			
	4	6	8	10
Total solids, %	2.85	4.25	5.60	6.85
Volatile solids, %	54.0	55.5	54.5	54.5
Volatile acids, mg per l	40	50	80	105
pH	7.0	7.05	7.15	7.20
Alkalinity, mg per l	2,310	2,975	3,620	4,300
Ammonia - N, mg per l	555	740	910	1,090
Grease, %	6.10	5.67	5.65	5.86

[a] Based on a 52-day period; average of 7-13 samples.

Summary.—The data obtained on the studies conducted at various detention periods and with sludges containing increasing solids content have shown that the fundamental concepts based on first-order reactions are applicable to sludge digestion.

IMPORTANT FACTORS INVOLVED IN HIGH-RATE DIGESTION

The reports of Schmidt[7] and Roy[6] have pointed out some of the variables that occur in high-rate sludge digestion that may be considered as factors limiting its application. These may be summarized as follows:

1. Biochemical end products
    a. Increased alkalinity
    b. Increased ammonia
2. Decreased grease destruction
3. Decreased volatile matter destruction.

In the writer's studies, we have encountered difficulty in maintaining good mixing in the digestion units when the digesting mixtures contain over 6% total solids. This is a factor which mitigates against the use of highly concentrated feed sludges, particularly sludges with low volatile matter content, and must be recognized if high-rate digestion units are to be properly designed and successfully operated. This factor brings into focus the importance of the fixed solids loading on digestion units as a design factor.

It has been recognized for some time that the digestibility of the volatile or organic matter in sewage sludge is a function of the volatile-fixed solids ratio. This matter was first brought to the attention of the profession by R. S. Rankin, F. ASCE in 1948[15] and is of such importance that his data are reproduced in Fig. 7. Experimental data obtained by Schmidt,[7] Roy,[6] and those in the current studies, have indicated that the volatile matter destruction during high-rate operation, using a 15-day detention period, is equivalent to the destruction obtained in about 35 days under conventional conditions, and the destruction obtained in 10 days under high-rate conditions is equivalent to that obtained in 30 days under conventional operation. An interpretation of these data in terms of high-rate digester operation is presented in Fig. 8.

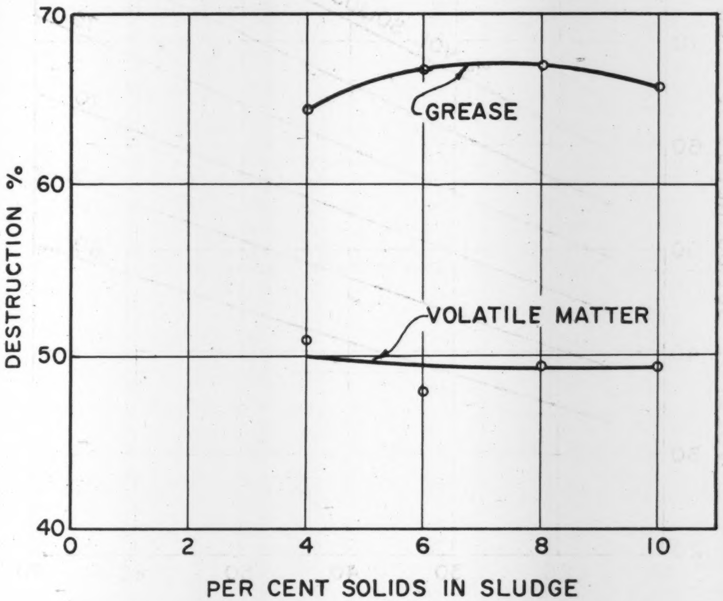

FIG. 6.—EFFECT OF SLUDGE CONCENTRATION ON DESTRUCTION OF ORGANIC MATTER.

From the information shown in Fig. 8, it is possible to estimate the solids content of mixed liquors in high-rate digestion units, when fed sludges with

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[15] Sewage Works Journal, by R. S. Rankin, Vol. 20, No. 478, 1948.

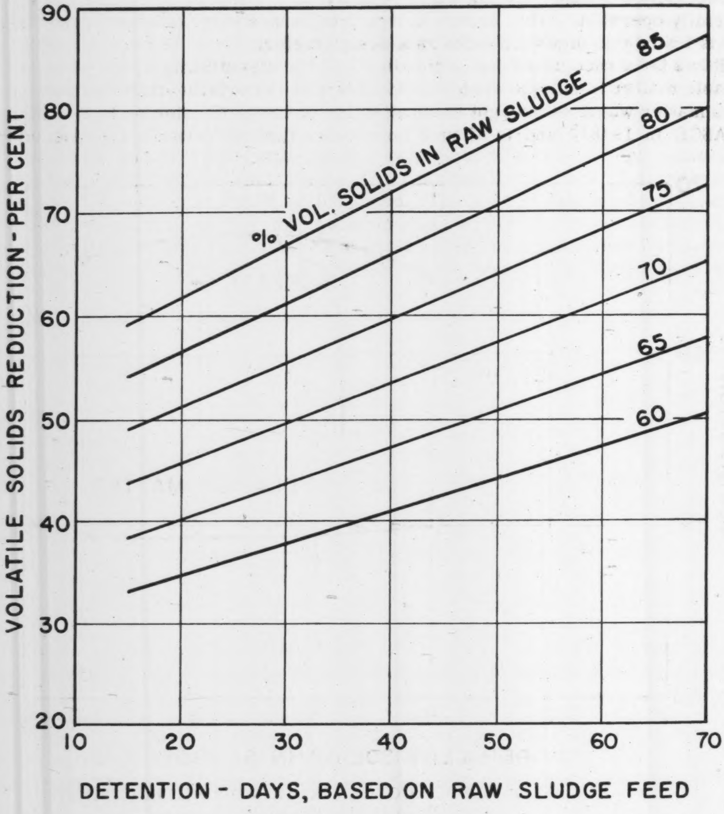

FIG. 7.—REDUCTION IN VOLATILE SOLIDS IN RAW SLUDGE FOR DETENTIONS FROM 15 TO 70 DAYS, T = 85° - 95° (REF. 5)

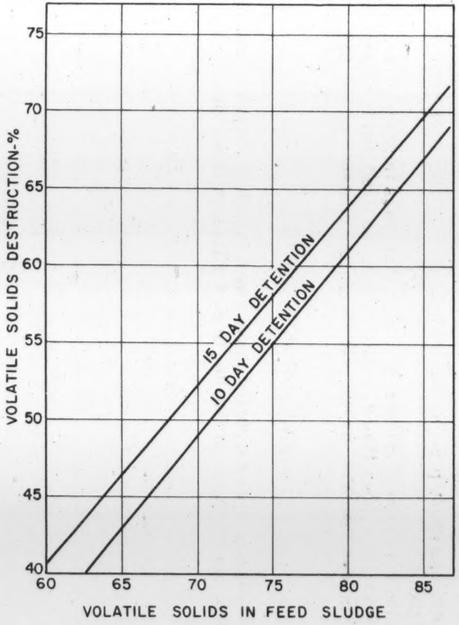

FIG. 8.—EXPECTED VOLATILE SOLIDS DESTRUCTION DURING HIGH RATE DIGESTION

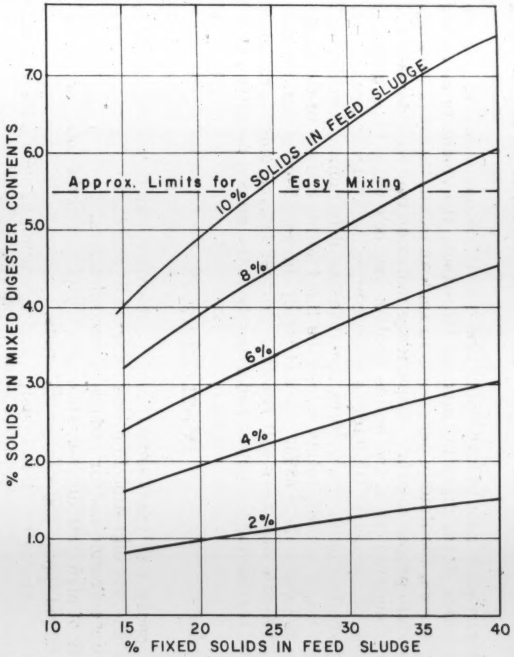

FIG. 9.—RELATIONSHIP BETWEEN SOLIDS CONTENT OF HIGH-RATE DIGESTERS TO FIXED SOLIDS IN FEED SLUDGE (15 DAY DETENTION)

varying percentages of fixed solids. This has been done for a unit operating on a 15-day detention period and the calculated relationships are shown in Fig. 9.

The major requirement in high-rate digestion is adequate mixing. As the percentage solids in the digesting mixture increases, the viscosity increases or, in other words, the fluidity decreases to such an extent that it is very difficult to maintain proper mixing. Experience indicates that considerable difficulty is encountered when the solids content reaches 6%. This difficulty points up one of the major limitations of concentrating sludges; from the information given in Fig. 9, the writers conclude that it is impractical to thicken sludges to more than 8% solids unless the fixed solids content of the sludge is less than 35% (volatile > 65%). To add highly thickened low volatile content sludges to high-rate digesters will result in process failure, unless provisions are made for increasing the power and capacity of the mixing facilities.

CONCLUSIONS

1. The concepts of unimolecular and first-order reaction kinetics can be applied to high-rate digestion to predict effects of increased organic loadings by sludge concentration and/or reduced detention times.
2. Mixing of high-rate digesters becomes a problem as the total solids content of the mixed liquor approaches 6% solids.
3. A major parameter in the design and operation of high-rate digestion units is related to the fixed solids content of the raw sludge.
4. The total solids content of the mixed liquor in high-rate digestion units is a function of the fixed solids in the raw sludge and the detention time in the digester.
5. Feeding of highly concentrated, high fixed matter content sludges to high-rate digesters can cause process failure because of difficulties resulting from improper mixing.
6. Because of problems related to mixing, it appears impractical to concentrate feed sludges to more than 8% solids, unless the fixed solids content of the raw sludge is less than 35%, or the volatile matter content exceeds 65%.

ACKNOWLEDGMENTS

This investigation was supported in part by a P.H.S. Research Grant, K-75, from the National Institutes of Health, U. S. Public Health Service.

DISCUSSION

F. SULZER.[16]—The authors suggest that the rate of digestion can be theoretically explained by applying the laws of chemical reactions to the equation

$$\text{"organic matter + bacteria} \longrightarrow CH_4 + CO_2\text{"}.$$

Noting that the equation corresponds formally to a binary (that is, bimolecular?) reaction and that the "bacteria" remains essentially at constant concentration, the authors conclude that ". . . the kinetics are the same as those for unimolecular reactions . . . . ."

Such a formulation suggests that complex biological systems might be analyzed in a simple fashion by arranging any type of reactants in an expression analogous to chemical reaction equations and by applying, consequently, the kinetic laws for chemical reactions in the homogenous phase to these "overall" reaction expressions. Such an explanation for the observed first-order reaction rate is, however, rather misleading. (The expression "first order reaction" is used in preference to the terms "unimolecular" or "monomolecular reaction" if the reaction mechanism is not definitely known to exclusively involve one molecular species.) A fundamental understanding of the removal kinetics can only be gained from a study of simpler systems than sludge digestion.

The reduction of volatile solids in sludge digestion can, in principle, be considered as a biological process of substrate removal (substrate utilization, respectively) by microorganisms. Microbiologists working with isolated cultures have been using so-called "washed cell suspensions" for a long time as simple systems to study substrate utilizations. With washed cell suspensions the metabolic processes involving specific compounds can be studied without interference from unknown substrate compounds and without the effects of growth which frequently complicate and obscure the kinetics of a substrate-microorganism interaction. In the following discussion it is tacitly assumed that, in the systems considered, the increase in active cell material during the substrate removal is negligible in comparison to the amount of active cell material present.

In most studies with washed cell suspensions, substrate utilization has been followed by indirect means (such as carbon dioxide evolution or, with aerobic systems, oxygen uptake). Data on such indirect measurements of removal rates are profuse and widely scattered throughout the biological literature. They, however, furnish only indirect evidence of the substrate removal process. In some instances, the substrate has been determined directly.[17] The experiments show that, usually, the rate of removal of simple, specific compounds is constant, in other words, the reaction rate is zero-order. In some cases, diminishing rates of substrate removal, depending on organism and substrate, have been observed, but this can be expected since interference by inhibitory metabolic products is possible. Enzyme adaptation mechanisms also may alter the rate. However, one is led to the conclusion that the removal of simple, specific compounds is, in principle, a zero-order reaction. This is in agreement with the laws governing enzymatic reactions (if the apparent Michaelis constant of the rate determining enzyme is assumed to be very small).

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[16] Asst. Prof., Dept. of San. Engrg., Univ. of North Carolina, Chapel Hill, N. C.

[17] "The Adaptability of Glucozymase and Galactozymase in Bacterium Coli," by M. Stephenson and E. F. Gale, Biochemical Journal, 1937, Vol. 31, pp. 1311-1315.

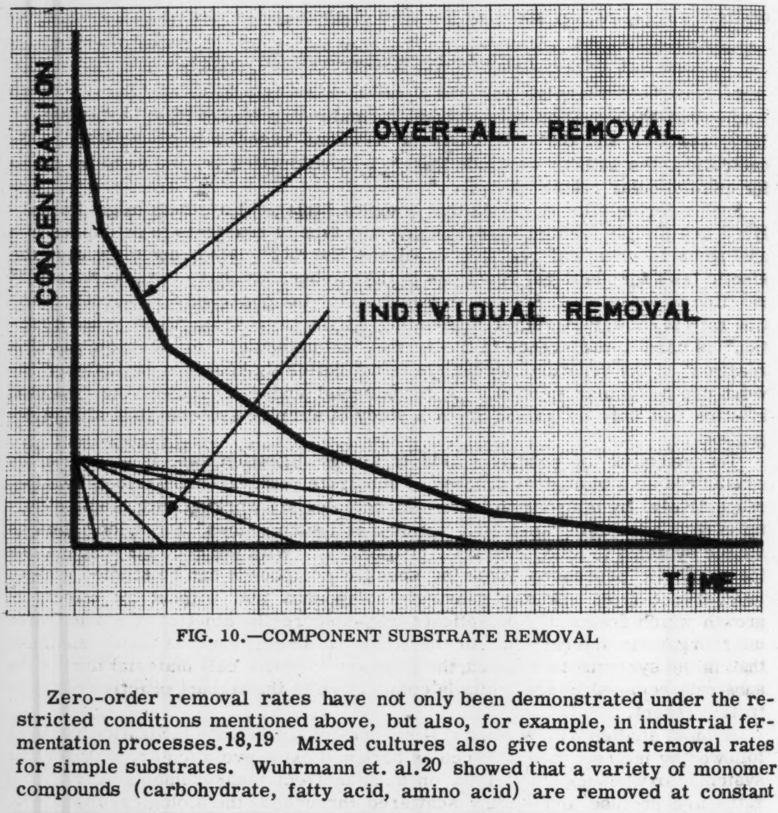

FIG. 10.—COMPONENT SUBSTRATE REMOVAL

Zero-order removal rates have not only been demonstrated under the restricted conditions mentioned above, but also, for example, in industrial fermentation processes.[18,19] Mixed cultures also give constant removal rates for simple substrates. Wuhrmann et. al.[20] showed that a variety of monomer compounds (carbohydrate, fatty acid, amino acid) are removed at constant

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[18] "The Effect of the Carbohydrate Nutrition of Penicillin Production by Penicillium Chrysogenum Q-176," by F. V. Soltero and M. J. Johnson, Applied Microbidogy, 1953, Vol. 1, pp. 52-57.

[19] "Itaconic Acid by Fermentation with Aspergillus Terreus," by V. F. Pfeifer, C. Vojnovich, and E. N. Heger, Industrial Engineering Chemistry, 1952, Vol. 44, pp. 2975-2980.

[20] "On the Theory of the Activated Sludge Process," by K. Wuhrmann and co-workers, Schweiz, Zeits. Hydrol., 1958, Vol. 20, pp. 284-310, 311-330.

rates by activated sludge. Data presented[21] by Weston and Stack indicate a zero-order BOD removal for wastes containing preponderantly sucrose (The authors explain this as the resultant of increasing metabolism due to growth and concurrent release of metabolic products by cell lysis, but there is hardly sufficient evidence for such an interpretation.). Similar evidence comes from laboratory soil percolation data,[22] which show that ammonium ion is oxidized to nitrate by an established soil microbe population at constant rate.

How then is the observed first-order reaction rate in sludge digestion to be explained? The same question can as well be posed for the aerobic processes of waste removal (trickling filter, activated sludge), where a similar first-order rate is observed. First, it should be recognized that the parameters used in measuring waste removal rates are unspecific (BOD, volatile solids, or similar units) and, therefore, express some over-all effect. Secondly, there are innumerably many compounds present in an ordinary sewage-type waste. If we assume that each compound is removed at a zero-order rate specific for the compound (and the microbial culture) and that these specific removal rates differ considerably, the over-all effect will approximate a first-order reaction. A simple model (Fig. 10) should illustrate the concept. Assume a substrate consisting of five different chemical compounds, present in equal concentrations (determined in some unspecific measure like BOD). As the individual removal rates vary widely, the over-all effect will simulate a first-order reaction or similar diminishing rate reaction (Fig. 10).

The first-order reaction rate in sludge digestion and similar biological processes can therefore be explained as the sum of a large number of zero-order reaction rates. It is realized that this concept needs further investigation and broader experimental proof, since many assumptions were made and the role of high-molecular substrates and suspended solids has not been dealt with.

C. E. KEEFER,[23] **F. ASCE.**—In his discussion of high-rate sludge digestion Clair N. Sawyer rightly indicates that the mixing of the contents of digesters becomes a difficult problem, especially when the concentration of the solids approaches 6%. When raw sludge is introduced into a digester containing digested material and the two materials are mixed together, it is not only difficult to mix intimately and thoroughly the two sludges, but when sludge is withdrawn day by day equal in amount to the raw sludge added, the sludge that is removed will contain considerable quantities of partially digested solids. This situation can be illustrated by the following example. Let it be assumed that a digester is filled with 100 units of digested sludge and that under ideal conditions of temperature, proper mixing, and so forth, raw sludge will digest in 10 days.

To simplify the computations let it be assumed that the percentage of organic matter digested day by day is directly proportional to the period of digestion, that is 50% of the organic matter is digested in 5 days and 100% in 10 days. Although this assumption is, strictly speaking, not correct, it does not differ from the actual facts so as to greatly change the results. Let it be assumed that at the beginning of each day 10 units of sludge are removed from the digester, 10 units of way sludge are then immediately added to the digester and the tank contents are thoroughly mixed. The following tabulation gives the percentage of partially digested solids in the sludge withdrawn day by day up to the end of 20 days.

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[21] "Prediction of the Performance of Completely-Mixed Continuous Biological Systems from Batch Data," by R. F. Weston and V. T. Stack, Conf. on Biol. Waste Treatment, Manhattan College, No. 24, 1960.

[22] "Biochemistry of Nitrification in Soil," by J. H. Quastel and P. G. Scholefield, Bact. Revs., Vol. 15, 1951, pp. 1-53.

[23] Deputy Sewerage Engineer, Dept. of Pub. Wks., Baltimore, Md.

Digestion Time, days	Percentage of partially digested sludge in the material withdrawn
1	9.0
2	16.3
3	22.2
4	27.0
5	30.9
6	34.0
7	36.5
8	38.6
9	40.3
10	41.6
15	45.4
20	46.7

These figures indicate that at the end of 10 and 20 days the sludge withdrawn will contain 41.6% and 46.7% of partially digested solids, respectively. Although these percentages are admittedly high because the digestion of sewage solids is more nearly a unimolecular reaction than one where the reaction rate is constant, the computations indicate that if daily additions of raw sludge are added to a digester filled with ripe sludge and the tank contents are then thoroughly mixed and sludge withdrawals are made equal in amount to the sludge additions, the sludge withdrawn will contain appreciable amounts of sludge that has not been thoroughly digested.

The foregoing difficulties associated with high-rate sludge digestion have been avoided at the Patapsco and the Back River sewage treatment plants in Baltimore, Md. by pumping the raw sludge into a small premixing tank, into which ripe sludge is drawn from the digester. These two sludges are thoroughly mixed for about 5 min, and the mixture is then introduced into the digester. Since each daily addition of raw sludge is pumped into the top of the digester and since the contents of the digester are not stirred, the rate at which the raw sludge moves downward to the outlet pipe depends on the amount of digested sludge that is removed and disposed of daily from the digester. The experiments that were conducted in Baltimore[24] using a digester with a working capacity of 180,000 cu ft indicated that satisfactory digestion could be obtained in about 10 days. It was found essential to thoroughly premix the raw and the digested sludges before introducing them into the digester so that each particle of raw material was brought into intimate contact with an ample quantity of digested solids. The best results were obtained when about one part, by weight, of raw volatile solids was mixed with one part of digested volatile solids.

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[24] "Effects of Premixing Raw and Digested Sludge on High-Rate Digestion," by C. E. Keefer, Sewage and Industrial Wastes, Vol. 31, No. 4, April, 1959, p. 388.

In view of the foregoing discussion, it is considered preferable to seed the raw sludge by premixing it with an adequate amount of digested sludge for several minutes in a small tank outside of the digester and then introducing the mixture into the digester. By following this procedure the raw solids can be thoroughly mixed with ripe sludge and there is no danger of drawing off sludge that has been in the digester for only a short period of time.

M. T. GARRETT, JR.[25]—The authors have stressed the value of first order reaction kinetics as a basis for predicting the effects of increased organic loadings or reduced detention times or a combination of the two. They have pointed out that at low detention times, approximately 3 days, the process fails and no longer conforms to first order kinetics. They have indicated that according to the overall equation,

$$\text{ORGANIC MATTER + BACTERIA} \rightarrow CH_4 + CO_2\,,$$

the reaction might be second order because of the two reactants. It was stated that since the bacteria are not consumed in the reaction, the reaction is actually first order in character.

Because the bacteria are not consumed, they must be a product of the reaction; the role of catalyst might be assigned to the bacteria and the equation written as

$$\text{ORGANIC MATTER} \xrightarrow{\text{Bacteria}} CH_4 + CO_2\,.$$

This form of the equation leaves only one reactant so that the reaction may be considered first order with respect to the organic matter.

If the concentration of the catalyst, that is, the bacteria, is important, the rate equation is

$$\frac{-dC}{dt} = k\,C\,(\text{Bacteria}) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

in which t is time (commonly measured in days) for sludge digestion, C is the concentration of digestible solids, and k is the rate constant. The concentration of bacteria may be combined with k to give a new rate constant k' so that,

$$\frac{-dC}{dt} = k'\,C \quad \ldots\ldots\ldots\ldots\ldots\ldots (1b)$$

For a continuously fed, mixed digester, the solids destroyed are equal to the rate constant times the solids in the digester. The equation for the operation is

$$\left(C_o - C\right)Q = k'\,C\,V \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which C_o is the concentration of digestible solids in the feed, Q is the rate of flow into and out of the digester, and V is the digester volume. The equation may be rearranged to read

$$\frac{C}{\left(C_o - C\right)} = \frac{Q}{k'\,V} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The progress of digestion is normally followed by the destruction of volatile solids. The volatile solids destroyed are readily measured. The digestible volatile solids remaining, C, is unknown so that Eq. 3 cannot be solved directly. To overcome this difficulty, let x be the concentration of undigestible volatile solids and C_V equal the concentration of volatile solids in the sludge. Then

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[25] Asst. Supt. of Sewage Treatment Plants, Houston, Tex.

$$C = C_V - x \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(4a)}$$

and

$$C_o = C_{V_o} - x \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(4b)}$$

Substituting these values in Eq. 3,

$$\frac{C_V - x}{\left(C_{V_o} - x\right) - \left(C_V - x\right)} = \frac{Q}{k'\ V} \quad \ldots\ldots\ldots\ldots \text{(5a)}$$

and

$$\frac{C_V - x}{C_{V_o} - C_V} = \frac{Q}{k'\ V} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(5b)}$$

When C_V is small, the equation approaches

$$\frac{C_V - x}{C_{V_o}} = \frac{Q}{k'\ V} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(6a)}$$

from which,

$$\frac{C_V}{C_{V_o}} = \frac{x}{C_{V_o}} + \frac{Q}{k'\ V} \quad \ldots\ldots\ldots\ldots\ldots \text{(6b)}$$

A plot of C_V/C_{V_o}, the fraction of volatile solids remaining, versus Q/V give a nearly straight line with an intercept at x/C_{V_o}, the undigestible fraction of the feed, and a slope of approximately 1/k'. The value of x/C_{V_o} may be determined by extrapolation or other more precise means. The value of k' should be computed from Eq. 5b because Eq. 6b is an approximation.

Data presented by the authors, by N. Sawyer and H. K. Roy,[26] and by W. N. Torpey,[27] F. ASCE are shown plotted in this manner in Fig. 11. The raw sludges were in the range of 75% volatile and temperatures were in the range of 85° to 98°F. A distinct difference is apparent in the fraction of undigestible solids in the primary activated sludge mixture as compared to the primary sludge. However, a difference in the rate constant, k', is not so obvious. The data of Mr. Torpey's are sufficiently scattered to allow quite a range of values of k' to be selected.

If C and C_o - C are measured in terms of gas production, then C_o - C is the gas produced in the digester, and C is the potential gas production remaining in the sludge withdrawn. Eq. 3 may be rearranged into the form,

$$k' = \frac{\left(C_o - C\right) Q}{C\ V} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(7)}$$

The value of k' is identical, except for the factor of 100, to the Digestion Index proposed by Mr. Torpey,[27]

$$\text{Digestion Index} = 100\ \frac{\text{Gas produced in digester}}{\text{Residual gas in sludge x detention time}}$$

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[26] Sewage and Industrial Wastes, by N. Sawyer and H. K. Roy, Vol. 27, 1955, p. 1356.

[27] Ibid., by W. N. Torpey, Vol. 27, 1955, p. 121.

For the data shown in Fig. 19, Mr. Torpey obtained values of the Digestion Index of 90% to 140% per day. The curve shown in Fig. 11 is for a k' value of 1.0 per day, or a Digestion Index of 100% per day, with an undigestible fraction of 48% of the volatile solids in the feed. This gives reasonable agreement with the data for the pilot plant and the treatment plant reported by Mr. Torpey[27] and also the data of Sawyer and Grumbling.

The results for primary sludge obtained by Messrs. Sawyer and Roy[26] indicate a rate constant of 0.84 per day on the assumption of continuous feeding. A somewhat lower rate constant, about 0.7, would produce the same degree of digestion with once a day feeding as was practiced in the experiments.

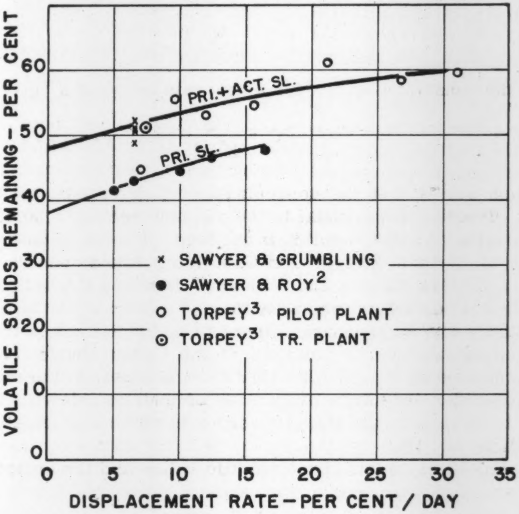

FIG. 11.—EFFECT OF DISPLACEMENT RATE ON VOLATILE SOLIDS REMAINING

The higher rate constant obtained with the activated sludge could be due to the great population of facultative bacteria in the activated sludge. Surely the fact that much of the digestible organic matter has been consumed in forming the activated sludge contributes to the higher undigestible fraction.

The data shown in Fig. 11 indicate good agreement with the first order reaction rate, Eq. 1a, so that for most practical purposes it can be considered to be valid. Because the undigestible fraction must be determined in order to use Eqs. 2, 3, and 5, there seems to be little advantage over the simpler approximation in Eq. 6b and the plot in Fig. 11 because the slope of the line is independent of the intercept or undigestible fraction for a given reaction rate k'.

The effect of the concentration of bacteria bears further consideration. The principal unknown is, of course, the concentration of bacteria. The population is composed of (1) those bacteria introduced with feed, and (2) those produced by growth on the organic matter within the digester. The quantity produced from the latter source are, in general, proportional to the quantity of organic matter destroyed, so that,

$$\text{(Bacteria)} = B\left(C_o - C\right) \quad \text{(8)}$$

If this is the principal source, then substitution in Eq. 1a gives

$$\frac{-dC}{dt} = k\,C\,B\left(C_o - C\right) \quad \text{(9)}$$

For a continuously fed digester the equation is

$$Q\left(C_o - C\right) = K\,C\,B\left(C_o - C\right)V \quad \text{(10)}$$

from which,

$$C = \frac{Q}{k\,B\,V} \quad \text{(11)}$$

Combining the constant B with k for a new rate constant k'' gives,

$$C = \frac{Q}{k''\,V} \quad \text{(12)}$$

This equation states that the concentration of digestible volatile solids remaining is directly proportional to the displacement rate, and independent of the concentration of volatile solids in the feed. This is in contradiction to the conclusions of Messrs. Sawyer and Grumbling based upon their studies, and the report of Messrs. Sawyer and Schmidt,[28] in which it was found that a given detention time produced substantially the same destruction of organic matter. Moreover, there was no pronounced trend to the variations in the data towards an increased destruction of volatiles at the higher sludge concentrations as would be predicted by Eq. 12. In order to look closer at the matter it is necessary to consider the large proportion of undigestible volatiles that it was necessary to assign to the sludge in order to obtain agreement with the first order equation, Eq. 1b.

Writing Eq. 12 in terms of the volatile solids and the undigestible volatile solids gives

$$C_v - x = \frac{Q}{k''\,V} \quad \text{(13a)}$$

Dividing all terms by C_{vo} and transposing gives

$$\frac{C_v}{C_{vo}} = \frac{x}{C_{vo}} + \frac{Q}{C_{vo}\,k''\,V} \quad \text{(13b)}$$

According to Eq. 13b the curves in Fig. 11 should be straight lines with slopes of $1/C_{vo}\,k''$. The data is sufficiently scattered that although there is no immediate confirmation of Eq. 13b, it would appear unwise to conclude that the data disprove it.

There are ample reasons for considering that the concentration of bacteria present is significant, the condition that leads to Eqs. 12 and 13b. First, the importance of seeding is well established. Second, the flow scheme of the activated sludge process has been applied to anaerobic digestion of industrial wastes[29] in order to accelerate the digestion by increasing the ratio of bacteria to digestible solids. If Eq. 12 is valid, the value of k" should be applicable to both high rate sludge digestion and other anaerobic digestion processes.

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[28] C. N. Sawyer and H. Schmidt, Journal, Boston Soc. of Civ. Engs, Vol. 42, 1955, p. 1.

[29] Biological Treatment of Sewage and Industrial Waste, by A. J. Steffen, Reinhold Pub. Corp., New York, N. Y., 1958, Chapters 1-12.

The lack of confirmation of Eq. 12 by the data in Fig. 11 may be due to the increased concentration of inhibiting substances formed when higher concentrations of sludge are digested. The authors found that the gas production decreased with increasing solids concentration in the sludge in the range of 4% to 10% solids. They considered both the reason stated previously and the possibility of loss of gas producing material in the concentration process. The reason is not clear, but they have shown that because of this and the mechanical difficulties of mixing the heavier sludges, concentrations above 6% solids, even with normal ash contents, should be used with caution.

So far the digestion process has been discussed from the standpoint of a single step process, and the agreement of the data with mathematical interpretations has been reasonable. However, it is well known that the process consists of at least two steps; the hydrolysis of complex organic matter to "volatile acids" and the metabolism of the volatile acids to methane and carbon dioxide by the methane bacteria. Under normal conditions the hydrolysis must be the rate controlling step since the concentration of remaining digestible solids is greatly in excess of the volatile acids concentration. Evidence of this can be noted from the data of Messrs. Sawyer and Roy.[26] The volatile solids in the feed sludge was 3.29%, of which 38% was undigestible according to Fig. 11, for an undigestible concentration of 1.25%. At the ten day detention time the digested sludge contained 2.99% solids at 56.4% volatile for a volatile solids concentration of 1.69%. This leaves a concentration of digestible volatiles of 1.69 - 1.25 or 0.44%. This is 4,400 mg per liter as compared to volatile acids of 110 mg per liter. Yet, if the metabolism of the methane bacteria is inhibited, the volatile acids accumulate and failure is imminent. In addition, if the rate of displacement of solids from the process exceeds the rate of growth of the methane bacteria, the process must fail.

So little basic research has been performed on the kinetics of the growth of the methane bacteria that even the maximum rate of growth must be inferred from sludge digestion studies. In a high rate digester the rate of growth (not maximum rate of growth) is equal to the rate of displacement, Q/V. It is obvious that the maximum rate of growth is at least slightly in excess of the greatest displacement rate at which stable operation can be obtained. On the basis of the studies by Mr. Torpey[27] the maximum rate is at least 0.31 per day. E. A. Cassell, A. M. ASCE and C. N. Sawyer have presented[30] evidence that at least two types of methane bacteria are significant. The one that developed last, presumably having the lower growth rate, was responsible for grease destruction and the metabolism the major portion of the volatile acids. However, no attempt was made to determine, from their data, the rate of growth of either group. Until sufficient information is obtained on the maximum rate of growth of the various methane bacteria and the conditions under which these rates obtain, the cause of failure at some particular high rate displacement cannot be ascribed with certainty to the maximum growth rate of the methane bacteria in preference to some unfavorable environmental condition.

Although sludge digestion is a biological process, the problems are so similar to problems of catalysis that it seems pertinent to call attention to

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[30] Sewage and Industrial Wastes, by E. A. Cassell and C. N. Sawyer, Vol. 31, No. 123, 1959.

C. N. Hinshelwood's[31] statement that: "There is no theory of catalysis. The only question is whether we understand catalytic phenomenon well enough to arrange them into a picture of which we like the pattern." Until sufficient information is obtained to formulate a sound theory, there will remain considerable room for personal preference of the various pictures of sludge digestion data that can be arranged.

CLAIR N. SAWYER,[32] and JAY S. GRUMBLING,[33] A. M. ASCE.—The contributions made by Messers. Sulzer and Garrett relative to describing the kinetics of anaerobic digestion in terms of first order reactions is appreciated. There seems to be general agreement that the gross digestion process produces a net effect that can best be described by first order or monomolecular reaction kinetics.

In spite of the many assumptions made by Mr. Sulzer in his discussion, it is agreed that zero order reactions are involved in digestion and are responsible for a significant portion of the total changes occurring during anaerobic digestion. In order for zero order reactions to occur, however, it is generally accepted that substrate concentrations must exceed particular levels. Before concentrations reach and exceed these levels, or after they fall below these levels, the reaction proceeds in accordance with monomolecular behavior. Fig. 12 is presented to illustrate these phenomena.[34] It is therefore likely that the straight lines shown on Fig. 10, denoting individual removal and representing zero order reactions from start to finish, actually become curves as they approach the abscissa although the net effect would be essentially as Sulzer has indicated in the composite curve for overall removal.

Mr. Garrett has extrapolated the mathematics of first order reactions in light of known phenomena, to arrive at some interesting conclusions. By applying his Eq. 6b to published data, separate curves are developed in Fig. 11 for primary sludge and primary plus activated sludges. The order in which the curves fall is logical, but the rate constants derived do not seem consistent with the nature of the sludges undergoing digestion. The writers do not believe that the population of facultative bacteria in activated sludge is sufficient to cause the difference in rate constants. The rate constants derived from W. N. Torpey's[35] F. ASCE, work have been obtained from data on a series of studies with a single pilot unit being fed "mine run" sludges for extended periods of several months. Because these data were not obtained from parallel studies using identical sludges, this undoubtedly accounts for the considerable scattering of points plotted in Fig. 11. The normal variations in sludge character received in day to day operation at sewage treatment plants is considered sufficient basis to raise serious questions on the derivation of rate constants from such data. The data from digesters being run in parallel would appear to provide a more valid basis for evaluating rate constants.

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[31] Journal of the Chemical Society, by C. N. Hinshelwood, London, England, 1939, p. 1203.

[32] Assoc. Engr., Metcalf and Eddy, Boston, Mass.

[33] Research Engr., Metcalf and Eddy, Boston, Mass.

[34] General Biochemistry, by J. S. Fruton and S. Simmonds, 2nd Ed., John Wiley and Sons, Inc., New York, 1958, p. 245.

[35] "Loading to Failure of a Pilot High-Rate Digester," Sewage and Industrial Wastes, by W. N. Torpey, Vol. 27, 1955, p. 121.

The writers do not agree with Mr. Garrett's interpretation of Eq. 12, "that the concentration of digestible volatile solids remaining is directly proportional to the displacement rate. . . ." Rather, they interpret the equation to mean that the concentration (or percentage) of digestible volatile solids remaining is proportional to the displacement rate as modified by the rate constant and independent of the concentration of the volatile solids in the feed. Two sludges with equal reaction rates will have essentially the same percentage of digestible solids remaining when operated at the same displacement rate, irrespective of the concentration of volatile solids in the feed. The writers see no disagreement with their findings and those of C. N. Sawyer and H. Schmidt[36] in studies involving the digestion of various sludge concentrations at definite detention times.

Sludge digestion is accepted to be a two stage process in which the second stage, involving the conversion of volatile acids and other simple derivatives of hydrolysis and oxidation-reduction to methane and carbon dioxide, is brought about by methane forming bacteria. From their limited observations of gas production from digesters batch-fed once daily, the writers are of the opinion that gas production is directly proportional to the bacterial population as long as the available substrate is plentiful and not inhibitory. The usual sequence of events immediately after batch feeding an operating digester is for the volatile acids content to increase, accompanied by an increase in gas production that reaches a maximum rate and remains at this rate until the volatile acid concentrations are reduced to normal levels. After volatile acids are reduced, gas yields become limited to, and directly related to the rate of volatile acid

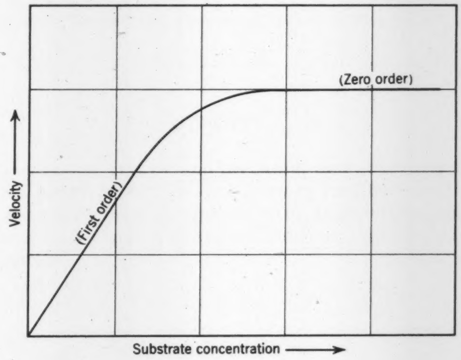

FIG. 12.—FIRST ORDER REACTION WHEN SUBSTRATE CONCENTRATION IS LIMITED AND ZERO ORDER REACTION WHEN SUBSTRATE CONCENTRATION EXCEEDS PARTICULAR LEVELS

formation. Methane formation can therefore be considered a zero order reaction having the characteristics of the curve shown on Fig. 12. Due to the restrictions of the first stage of digestion in the first few hours after feeding, methane formation assumes the identity of a first order reaction and then behaves as a zero order reaction until again limited by substrate concentrations.

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[36] "High-Rate Sludge Digestion," Journal, Boston Soc. Civ. Engrs., by C. N. Sawyer and H. E. Schmidt, Vol. 42, 1955, p. 1.

Mr. Keefer has pointed out some of the limitations of high rate digestion as regards internal mixing within a digester. His proposal to intimately mix raw and digested sludges in proper ratio before introducing fresh solids into a digester has merit and deserves continued study at other locations so that the practice can be evaluated on a wide variety of sludges.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3227

RUSSIAN WATER SUPPLY AND TREATMENT PRACTICES

By V. J. Calise[1] and W. A. Homer[2]

With Discussion by Messrs. K. J. Ives; and V. J. Calise and W. A. Homer

SYNOPSIS

Russia has many lakes, rivers, and streams, and generally has an abundant water supply. This paper reports on a survey of facilities in Russia and Eastern Europe with emphasis on municipal treatment practices and supplies. The technical literature and reports from visitors to Russia provided the main sources of information.

INTRODUCTION

Purpose of Paper and Sources of Information.–**This paper is the result of an attempt to survey water-treatment practices in Russia and Eastern Europe with emphasis on municipal practices and water supplies. In preparing this paper, the writers had available three sources of information:**

1. **The technical literature from Russian periodicals, almost all of which required original translation and review for preparation of this paper. More than 150 literature references were reviewed, of which approximately thirty technical articles were translated into English.**
2. **Information from American and other visitors to Russia.**
3. **Information directly from Russian experts and authorities in this field. Direct communication was established with a number of these specialists.**

Note.—Published essentially as printed here, in March, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2407. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The technical literature and reports from visitors to Russia provided the main sources of information.

Because of the relatively limited amount of up-to-date literature available and the failure of direct communication and a personal visit to Russia, (which had been scheduled for the summer of 1958 and had to be cancelled because of business commitments), the information contained herein is not complete nor does it necessarily represent the most up-to-date thinking and practices in all areas covered in this report. Nevertheless, the picture drawn from the available data of the state of the art in this field in Russia is, in the main, an accurate indication of what is to be found there.

Literature and data available from East European countries was much more limited and a study of this literature showed less progress than in Russia.

Various Aspects Investigated.—It was the purpose of the writers to obtain data concerning:

a. The nature of various water supplies in Russia and Eastern Europe.

b. The general prevalence and use of, as well as the relative emphasis on, various water treatment processes, equipment designs and chemical results for application in treatment of municipal (potable) supplies.

c. The engineering and economic evaluation of the various processes cited above, with respect to -

(1) Chemical costs and availability of various chemicals;

(2) Material costs and availability of construction materials; and

(3) The effect of centralized standards (if any) on equipment designs, and treatment results for various water uses (potable, boiler feed, etc.) and the prevalence and use of centralized public health standards for total dissolved solids, sterilization, total hardness, alkalinity, et al.

NATURE OF WATER SUPPLIES IN RUSSIA

In general, Russia is a land of many lakes, rivers and streams and has an abundant water supply in virtually all areas. Larger cities are located close to fairly abundant surface water supplies. Industry, including central station utilities, purchases water from municipal water treatment plants which are generally of large size and, in the main, are conventional sedimentation-sand filtration plants. Water treatment facilities such as power and steam utilities are planned for centralized design and control. The use of underground aquifers is not as prevalent in the U.S.S.R. as in Europe and in the United States. Table 1 shows five typical water supplies available in the Soviet Union. According to data published by the Hydrometerological Service, "low mineral" content waters of the type in Col. 2, 3 and 4 in Table 1, in which bicarbonates predominate over sulfates and chlorides, constitute about 80% of all rivers and lakes in the U.S.S.R. The high solids waters shown in Col. 5 were sulfates and chlorides predominate over bicarbonates are concentrated in the Donbas, Azov Sea, and Kazakhstan regions.

The frequent use of the word "demineralization" in Soviet literature as applied to partial solids reduction by lime treatment and split stream processes, as well as to full "demineralization," emphasizes the fact that the general run of high-hardness, high-bicarbonate waters in the U.S.S.R. lend themselves well to substantial solids reduction (usually 60-70%) by simple lime treatment or split-stream treatment. As with most surface supplies, seasonal variations introduce problems of dosage control and, in the case of split stream treatment, may reduce plant design capacities.

The practice of locating power plants and industries near the source of coal or other raw materials often "fixes" the water supply and requires the use of rather complex treatment plants to render the raw water suitable for use as makeup to high pressure boilers. The nature of the water supplies has also been a factor in the design of high pressure boilers (1,600 to 2,700 psig) resulting in virtual standardization in government prescribed multi-stage, multidrum boilers with two and three stages of evaporation; and also resulting in requirements for steam washing with very high relative amounts of feedwater or condensate.

GENERAL INDUSTRIAL WATER TREATMENT PRACTICES

The heavy industries in the U.S.S.R. are located, of necessity, in the same general regions as the electric-generating facilities. Many steel and metallurgical plants were moved during World War II to the Ural Mountains, where a heavy concentration of industry still remains. Much of the oil and chemical industry is concentrated in the Donbas-Black Sea area. The nature of the water supplies is similar to those shown in Table 1.

TABLE 1.—TYPICAL NATURAL WATERS OF THE USSR

Constituents (1)	Low (2)	Medium (3)	High (4)	Medium-High (5)	Very High (6)
Total Hardness (ppm as CaCO)	125	268	445	715	1070
M Alkalinity[a] (ppm as CaCO)	108	232	268	268	320
Ratio Ca to Mg	2.47	2.01	4.00	1.65	2.34
Sulfate SO_4 as $CaCO_3$	17.5	35.5	176	450	750
Chloride Cl_4 as $CaCO_3$	17.5	26.5	90	357	535
Silica as SiO_2	6	10	16-18	20-25	25-30

[a] M Alkalinity in all cases is bicarbonate alkalinity.

These surface supplies are treated by the various methods described subsequently according to the quality requirements of the particular industry. Most of the industrial plants built since 1950 have centralized facilities for water treatment. A typical metallurgical plant would treat a surface supply by coagulation and filtration to provide the bulk of its requirements for cooling water and other general uses. The remainder would be softened, usually by ion exchange, and deaerated for use in steam boilers, evaporative cooling of furnace walls, or for other high temperature cooling purposes.

More recently (as of 1960), the practice of centralized deaeration of all softened water, with regenerative cooling of the softened water going to distant locations, has become popular, particularly in steel plants.

The writers were unable to find extensive information concerning the treatment of process water for chemical plants or cooling water.

Many chemical and paper plants in the U.S.S.R. use large quantities of demineralized water in their process as well as for boiler feedwater, makeup treatment to high pressure boilers up to 1,650 psig and 60% makeup. One source indicated that demineralized water was used by the Voronezh Radio Component Works in the production of various electronic components. The writers suspect that the relatively high cost of anion resins and the lack of experience with demineralization has restricted its use for the present to critical applications such as makeup to very high pressure central station boilers and electronic or transistor manufacture.

MUNICIPAL WATER TREATMENT

Coagulation and sand filtration of the relatively abundant surface waters appears to supply the bulk of potable water needs for the large cities. The equipment used for this purpose, is, for the most part, conventional coagulation basins with about 4-hr detention, followed by sand and gravel gravity filters. The usual coagulants are used (alum or iron salts) with lime for pH correction. Chlorination is not as widely used as in the United States, but recent trends and the development of gaseous chlorine-feeding equipment indicate a future increase in the use of chlorine.

Chlorine usage is apparently limited because of high gaseous chlorine and sodium hypochlorite cost as well as lack of standards, such as A.P.H.A. or U.S.P.H.S., relative to sterilization by use of chlorine, ozone, or other materials and methods. Organic and B.O.D. reduction by simple clarification and filtration is generally employed.

Improvements in Coagulation Techniques.—The "divided coagulation treatment" of potable water supplies is not unknown in the United States and in Europe, but Russian water-treatment specialists have investigated this method on a large scale and are recommending its use, where applicable.

The "divided coagulation treatment" (sometimes known as concentrated dosage method) consists of dividing the raw water into approximately equal parts and feeding the total required coagulant dosage into one stream. This is comparable to the Hoover split-line treatment so successfully employed over the years in many municipal plants in the United States (1).[3]

After the normal flash mixing and coagulation process, the two streams (one stream untreated and the other with double the normal dosage) are recombined in the sedimentation basins where the normal sedimentation process takes place.

Several tests at different locations (2) (3) have apparently indicated that at low temperatures (1°C to 12°C) the divided method of treatment has some definite advantages with respect to shorter settling time and better sedimentation coefficient.

At higher temperatures (12°C to 20°C) the beneficial effect of divided treatment is not marked, although slight advantages still appear as noted in Fig. 1.

Improvements in Equipment Designs.—Using the conventional basins (4-hr detention) considerable savings in area and concrete were claimed by designing two-floor settling basins. The first such design at the Gorkii Water Supply was a series design, in which the water flowed into the lower section, along the length of the section and reserved direction flowing back along the upper floor to the clear water outlet.

A later design at the Moscow Water Works was a parallel-flow type in which sedimentation took place simultaneously in both upper and lower sections, the flow being in one direction only.

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[3] "The investigation of Two-Floor Water Conducting Settlers," by Yu. B., Bagotski, E. V. Sizova, Vodosnakzhemie i Sanit. Techn., No. 4, 1956, pp. 6-9.

Viewing and sampling tubes were provided through the floor of the top basin for checking performance of the bottom basin as shown in Fig. 2.

Extensive tests were made on the installation at the Moscow Water Works, which has a capacity of 6,900 cu m per hr. The tests, including a chloride-tracer test, indicated that the performance was comparable to the more conventional basins in use at the plant. A comparison of the two-floor basin with a single floor basin of the same capacity is shown in Table 2.

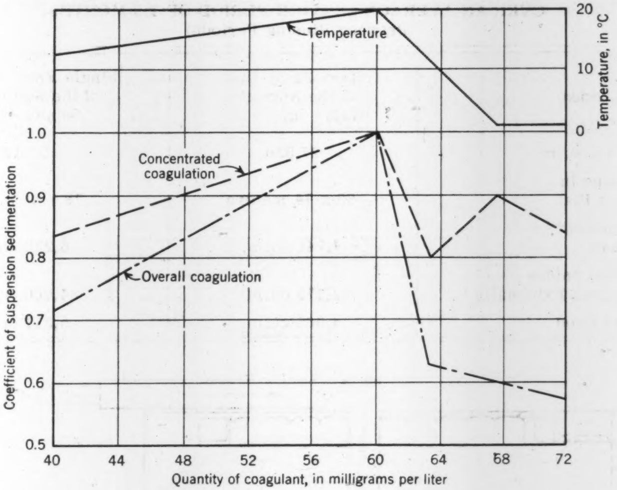
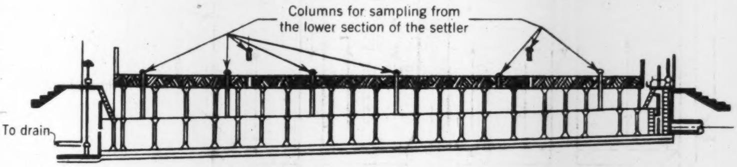
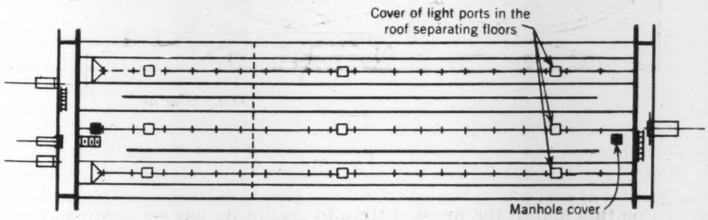


FIG. 1


Columns for sampling from the lower section of the settler
To drain

(*a*) LONGITUDINAL SECTION


Cover of light ports in the roof separating floors
Manhole cover

(*b*) LAYOUT OF ONE SECTION

FIG. 2

An upflow and contact type coagulation-filtration unit is being used at some municipal and industrial installations. Further development work is being carried out on this unit at the present time to enable its use on medium turbidity waters (over 150 mg per l). This unit will be described in more detail subsequently.

TABLE 2.—COMPARISON OF THE SINGLE AND TWIN-FLOOR SEPARATORS OVER AN AVERAGE SERVICE PERIOD OF 1.5 MONTHS
(Between Rinsing Periods)

Index	Two-Floor Unit of the Northern Water Supply Sta.	Single Floor Separator of the Stalin Water Supply Station
Volume in cu. m.	27,330	27,120
Dimensions in Layout Plan	63 x 74 meters	78 x 80 meters
Area Occupied by Settlers	4,741 sq.m.	6,250 sq.m.
Cubic Dimensions Measured Externally	32,175 cu.m.	34,000 cu.m.
Concrete Used	4,963 cu.m.	5,311 cu.m.

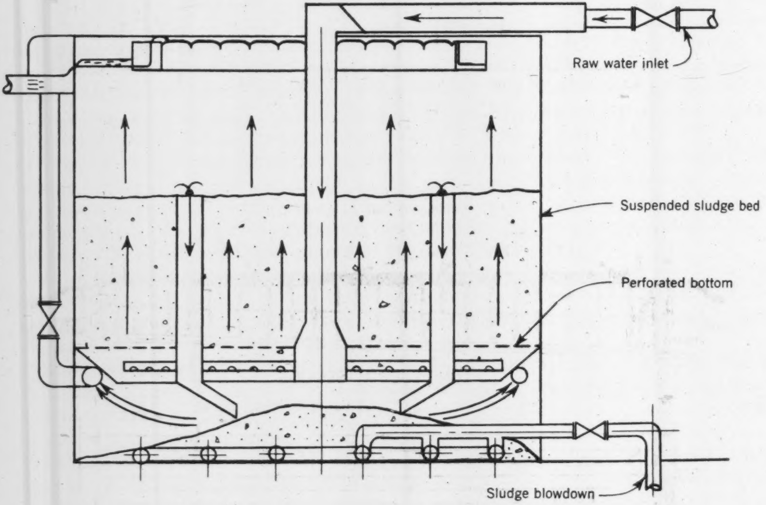
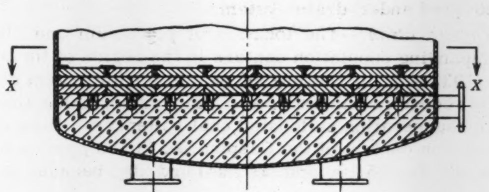


FIG. 3

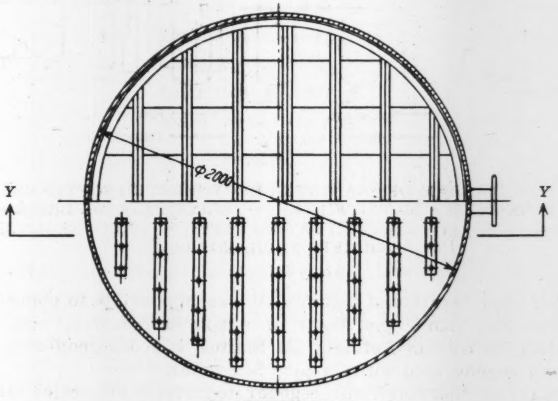
Considerable development work on high rate, suspended-sludge-type clarifiers was done and a few full-sized units were installed at several municipal installations in 1953-54. The operating experiences with these units were so unsatisfactory that all further work along this line was abandoned until very recently. Fig. 3 is a typical design.

BREAKDOWN OF TREATMENT PRACTICES BY PROCESS AND EQUIPMENT

Ion Exchange.—The use of ion exchange for water treatment is quite extensive in Russia and the Eastern European countries, particularly for softening on the sodium cycle. The early use of natural ion exchangers (zeolites) was accompanied by investigations of other natural exchangers or adsorbents (4), but widespread use of ion exchange for water softening was dependent on development of synthetic cation exchangers. The sulfonated carbon type of cation exchanger was apparently put into industrial use in the late 1940's or early 1950's and is still the only industrial synthetic cation exchanger in general use.



(*a*) SECTION *Y–Y*



(*b*) SECTION *X–X*

FIG. 4

Cold Sodium Cycle Softening.—The design of cold sodium cycle ion exchange units naturally parallels development of this process in the United States and Europe, although several interesting differences are noted. The later designs include upper and lower distributing systems (translated literally as "drainage plates") which are shown partially in Fig. 4. Upper distributors were of various designs in the early units but later designs were fairly standardized with a flat plate type using standard "caps" or nozzles designated by VIT (5).

The under drainage systems utilized various distributors such as header lateral type, with quartz sub-fill, all of which was covered by the lower cover plate. It is not clear what purpose was served by the cover plate, since practice in the United States for sodium-cation exchangers places the ion exchanger directly on the sub-fill. Considerable breakage was experienced with these cover plates due to sudden pressure rise under the plate during backwash (5). Some experimental designs used porous, inert plates laid directly on a concrete fill, in which was imbedded the header-lateral distributors. The experiments claimed a 10% increase in permissible resin volume with an increase of pressure drop of less than 3% as compared with an identical exchanger with a conventional, covered under-drain system.

Sea Water Regeneration.—The location of petroleum and chemical industries and accompanying population centers in the region of the Black Sea led at an early date (1932-40), to the use of sea water for regeneration of natural zeolites. More recently, (6) tests have been made, using the waters of the Caspian Sea for regenerating sulfonated coal softeners. Because of the relatively low sodium ion content of the Caspian Sea (3,180 ppm as Na^+) as compared to the Black Sea (5,100 ppm as Na^+) and also because of the relatively higher molar concentration of calcium, it was impossible to obtain an effluent with hardness less than 5 ppm. For this reason the industrial softening unit at the Bakiniskii Petroleum Refinery IM Stalina was designed with a primary softening unit regenerated with Caspian Sea water.

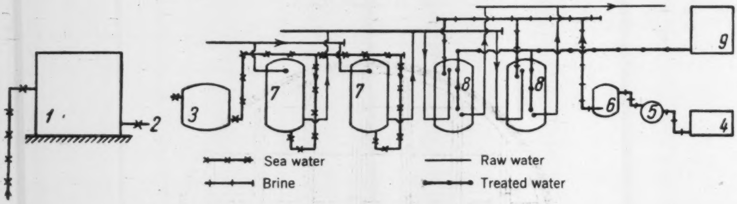


FIG. 5.—FLOW DIAGRAM OF SEA WATER REGENERATED SOFTENERS. (1: SEA WATER TANK; 3: SEA WATER FILTER; 4: BRINE TANK; 6: BRINE FILTER; 7: SEA WATER REGENERATED CATION UNITS; 8: BRINE REGENERATED "BARRIER" EXCHANGERS.)

A secondary or "barrier" unit, regenerated with commercial salt, was used to remove the residual hardness. Fig. 5 shows a flow diagram of this plant. The overall reduction in use of commercial salt was on the order of 20 to 1, although reduced capacity of the ion exchanger regenerated with Caspian Sea water (less than 1/3 the normal capacity) resulted in a considerably larger initial expenditure for equipment.

Other Ion Exchange Processes.—The use of ion exchangers for hydrogen-sodium blend systems and complete demineralization was limited until very recently by the lack of good, stable, low cost synthetic ion exchangers. In 1956 one or two demineralizing plants were installed for treating makeup to high pressure boilers and some experimental work was done concerning the potability of demineralized water using the synthetic resins available. Table 3 indicates the types of synthetic resins available in the U.S.S.R. at the end of 1957.

Cold Processing Softening.—The classical method of softening and alkalinity reduction using lime, supplemented with soda ash, caustic soda or phosphates, has been well-known and extensively used in Russia and in Eastern European countries for many years. The continuous-flow type of equipment, with external sludge recirculation, is widely used in Russian industrial plants and in some cases in municipal plants, although relatively long detentions require extensive equipment outlays. This is similar to German and European practices in the late 1940's and early 1950's.

There has been no widespread usage of the "high rate" upflow type of softening and clarification equipment such as sludge blanket or sludge (slurry) recirculation designs which have been in use in the United States for many years. An early design installed at several locations in the U.S.S.R. is shown in Fig. 3. This unit operated on the principal of a suspended sludge bed supported on a perforated floor. This design appears to be comparable to the suspended sludge-blanket units used in the United States. However, there is no indication of any original work in Russia, Europe, or elsewhere on the concepts and ideas of solids-contact in clarification, coagulation and lime softening through the use of sludge or slurry recirculation by internal pumping as used so effectively in units reduced in the United States.

TABLE 3

Ion Exchanger	Type	Availability
Sulfonated Coal	Cation (Low Capacity)	Standard Industrial
Wofatite R	Cation (Low Capacity)	Small Quantities
EDE-10P	Anion (Intermediate Basicity)	Standard Industrial
PEK	Anion (Strong Base)	Experimental
AV-15	Anion (Strong Base)	Experimental
AV-16	Anion (Strong Base)	Experimental
AV-17	Anion (Strong Base)	Experimental
Wofatite L-160	Anion (Weak Base with Some Capacity for Silica Removal)	Imported
AN-2F	Anion (Weak Base)	Standard Industrial

Later Russian designs incorporated a small clear-water collector in the bottom section to force sludge from the suspended bed into the bottom sludge-removal section. Numerous operating difficulties later caused this design to be abandoned. Apparently this was due to plugging of the perforated bottom and is better described by the authors of a Czech survey (7):

> "In actual service at the plant the perforated bottom proved to be a source of much difficulty which adversely affected the operation of the entire equipment. . . . an indication of the extent of these deleterious phenomena has been given by Candidate Varnelli from the Vodgeo Institute when checking the performance of the clarifying units of the Sludovski and Kibkshev Water Supply Systems in Gorkii. There were instances of the

obstruction of the perforated bottom with packed sediment to the extent of 40-50% of the area of the bottom. The layer of deposit on the perforated bottom reached as high as 1-1.2 meters. In addition, considerable obstruction was observed in the circulating area under the bottom."

Since the bulk of the surface waters are relatively high in hardness and alkalinity, and also because many industries and utilities purchase water from municipal treatment plants, partial softening and silica reduction is practiced in many municipal treatment plants.

V. M. Kyvatkoskii and L. M. Zhivilova (2) have done considerable experimental work in determining control limits, predicting dosages, and, in general, investigating the effects of pH, coagulants, and magnesia on softening and silica reduction.

Although similar investigations have been conducted in the United States, one interesting aspect of their work was the reduction of silica from 10 ppm to 15 ppm to less than 1 ppm, using raw waters with varying alkalinity and hardness. Several conditions were established as being necessary for complete silica removal:

1. Control of pH in the range of 10.1 to 10.3;
2. Sufficient magnesia present (in the ratio of 15 ppm of Mg to each ppm of silica removed), either by precipitation from the raw water or by addition of Mg ion; and
3. Sufficient contact with previously formed and recirculated magnesium sludge.

Several difficulties are noted in applying the correct treatment:

a. Control of the process by means of acidimetric titration was not reliable, since the actual OH^- concentration as determined from the pH measurements differed considerably from the OH^- concentration as determined by titration.

b. With certain low alkalinity raw waters, it was not possible to maintain the optimum pH for silica reduction without preducing an unstable effluent (that is, a decrease in the alkalinity of the treated water of 5 ppm to 25 ppm in passing through marble filters) although in many cases, increasing the coagulant dosage by 40 ppm to 60 ppm corrected the instability of the treated water to less than 5 ppm.

The Combination Clarifier-Filter.—A simplified, condensed method of clarification and filtration was developed and tested in 1953-54 at several municipal and industrial sites (Moscow, Leningrad, Cheliabinsk, and Gorkii). These tests indicated that with certain water supplies, the clarified and filtered effluent from the "contact clarifier" was equal to the effluent from conventional sedimentation and filtration equipment. The average values of water quality apparently satisfied the requirements of GOST (All-Union States Standard Committee).

The design of the "contact clarifier" is shown in Fig. 6. The raw water is treated with coagulant directly ahead of the inlet distributor. The water is distributed by header-lateral type pipe distributors at the bottom of the sand and gravel bed and flows upward through the filter media. Clarified and filtered water is collected at the top of the clarifier. All of the design data are not available (8) but from the area and flow figures given for a contact clarifier in operation at the Yuzhnoya Water Station of Leningrad, the writers calculate an upward flow rate of approximately 7 gpm per sq ft. of area. The graded filtered media depth is about 8 ft 6 in. and the total detention time is about 12 to 16 min.

It is stated that the equipment is not suitable, at present, for operation on water supplies with turbidities in excess of 150 mg per l. Early designs also encountered problems with shifting of the various grades of filter media, which were corrected by better distributor arrangements. Table 4 is a comparison of the contact clarifier with conventional equipment operating on the same water supply.

Unfortunately, no information concerning length of run between cleanings, methods used for cleaning sludge from the filter media, or frequency of replacement of filter media is available (8), but information from the Czech paper on high rate clarifiers (9) indicates that during the spring floods of 1956, the several contact clarifiers installed in the U.S.S.R. were unable to cope with the high turbidities encountered. Wash-water requirements were 28% or more of the total water treated and runs between washings were shortened to the extent that the systems often became inoperable.

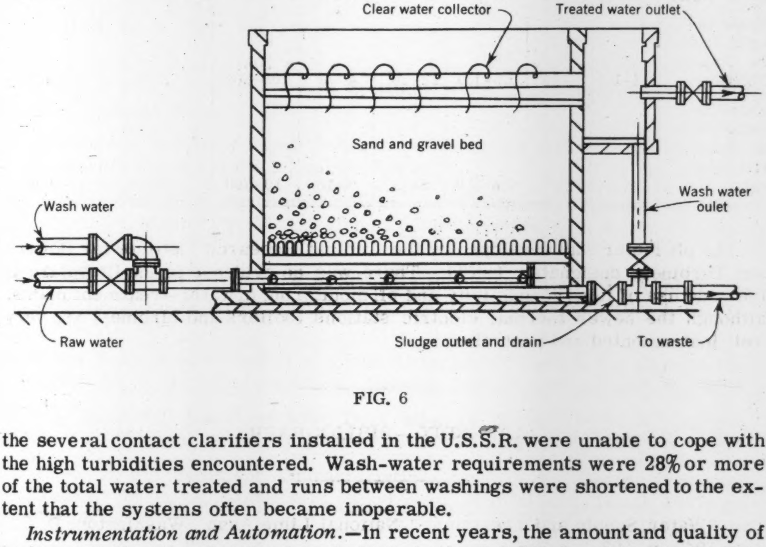


FIG. 6

Instrumentation and Automation.—In recent years, the amount and quality of instrumentation in water treatment plants in the U.S.S.R. has increased. This is particularly noticeable in the rather complex demineralization plants constructed for makeup to high-pressure boilers (10) (11) (12). Flow-metering instruments are provided in the main stream where required, and in the regenerant lines as well. Quality control instruments (conductivity, pH) are also used to check effluent and regenerant conditions, although the continuously monitoring type of instrument with automatic control and alarm contacts was not in evidence. No silicometers or other continuous analyzing equipment have yet been developed.

Hydraulically operated valves, manually controlled, are used in many of the larger plants, but the completely automatic ion-exchange units which are common in the United States are not found, indicated, or described anywhere in the literature or by visitors to the U.S.S.R.

Much of the instrumentation is fairly standardized and is manufactured and supplied by a particular industrial group. As an example, the water meters used for the demineralizing plant at one of the Mosenergo Stations were described as "Salt Meters TsLEM," a type widely used in the chemical industry.

TABLE 4.—OUTCOME OF PURIFICATION WITH CONTACT CLARIFIER

Type of Water	Cloudiness, Coloration Indices Mean Value Per Cycle				Bacteriological Data			
	Mean Conditions		Flood Conditions		Mean Conditions		Flood Conditions	
	Cloud MG/L	Color	Cloud MG/L	Color	Gastr. Bacter. Index	Bact. Count Per ML	Gastr. Bact. Index	Bact. Count Per ML
River Water	6.4-9.8	22-72°	16.7-215	42-115°	30-3000	200-2000	200-200,000	150-12,000
Water Issuing from the Contact Clarifier	0.4-1.5	9-18°	0.5-1.5	12-18°	4-72	700-800	7-2000	7-3000
Water Issuing from the Standard Apparatus	1	11°	0.6-1.8	8-19°	4-16	10-590	4-1400	6-4000

The pH meter was developed by the Central Research Institute for Boilers and Turbines, designated TsKTI. There was no evidence in the literature of the wide use of flow, conductivity and pH monitoring of water-treatment plants, although the newer thermal electric stations (boilers and turbines) are very well instrumented and controlled.

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DISCUSSION

K. J. IVES,[4] M. ASCE.—The principle of filtering upwards from the coarse through fine media has attracted the attention of Soviet water engineers, as indicated in the description of the contact clarifier given in the paper. However, it has the disadvantage that the media cannot be too fine at the surface or it will be expanded by the upflow.

This has been overcome by the development of the biflow filter at the Pamfilova Academy of Municipal Economy; this filter is referred to as the AKX filter. In the AKX filter the draw-off pipe for filtered water is located in the upper layer of sand, and raw water enters the filter from under the gravel base and from the sand surface and "biflow" is established downwards through the top few inches of sand and upwards through the bulk of the graded bed. The bed is cleaned by upwash in the normal manner except for a supplementary upwash for the top layer. Extensive details of the theory, experimentation, development, and operation of these AKX filters and contract clarification are set out elsewhere.[5] Operational results indicate the contact clarifier is equally effective (often superior) for the removal of zooplankton and phytoplankton, as conventional treatment plant, and average reduction in bacteria (colony plate

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[4] Lecturer in charge Pub. Health Engrg. Research, University College, London, England.

[5] "Contact Clarification for Water Purification," by D. M. Mints, Academy of Municipal Economy K. D. Pamfilova, Moscow, 1955.

count) was about 71%, compared with 74% for normal filtration. The problems of displacement of the filter layers are dealt with a commentary on the rational design of the distributing system arrangement. By using "suspended layer stabilizers" upwash rates of 21 in per min give regular washing with no strata displacement. Table 5 gives a comparison of the average operating characteristics of contact clarifiers, AKX filters, and normal rapid filters. Costs of construction and operation are given in the same publication.

TABLE 5.—COMPARISON OF AVERAGE OPERATING CHARACTERISTICS OF CONTACT FILTERS

Operation (1)	Units (2)	Contact Clarifier (3)	AKX filter (4)	Rapid filter (5)
Design filtration rate	gpm per sq ft	2	≯ 5	≯ 2.5
Maximum filtration rate	gpm per sq ft	2	≯ 6	≯ 3
Basic washwater rate	in per min	20	21	16
Wash duration	min	8	6	5
Top layer washwater rate	in per min	0	8	0
Top layer wash duration	min	0	1	0
Flushing rate through drainage pipes	in per min	0	13	0
Flush duration	min	0	2	0
Initial filtrate to waste : duration	min	5	0	0
Time out of operation	min	28	24	20
Filter run	hr	20	12	12
Operating cycle; run + wash	hr	20.47	12.4	12.33
Mean annual coagulant dose (anhydrous salt)	mg per l	20	25	25
Mean annual lime dose	mg per l	0	6.6	6.6

Many existing filter plants have been reconstructed to the AKX pattern, and practical details of such reconstruction have been given by M. M. Andriashev.[6] The operational results of such a reconstructed AKX filter with a normal rapid filter have been published by L. S. Kruglov.[7] His results are summarized in Table 6. The turbidity of the AKX filtrate averaged 2 mg per l (silica scale) rising to approximately 3 mg per l at the end of the run.

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[6] "Scientific Work, Information on Sanitary Technology," Academy of Munic. Economy K. D. Pamfilova, 1951.

[7] Vodosnakzhemie i Sanit. Techn. No. 10, 1956, p. 15.

No doubt following on the Soviet work, Polish engineers have given an account[8] of experimental work with contact clarification and have tabulated the results of operation of the contact clarifier.

In addition to their very practical studies on contact clarification and AKX filters, the Russians have made some interesting contributions to filtration theory. The most outstanding of these are by D. M. Mints[9] and by H. Ornatskii, G. Sergeyev, and J. Sechtman.[10] These theories, together with those of Mackrle in Brno, and of Ives at Harvard and London, represent the only rational theories of the filtration of suspensions through deep granular filters.

TABLE 6.—SUMMARY OF KRUGLOV'S RESULTS

Operation (1)	Units (2)	AKX (3)	Normal (4)
Rate of filtration	gpm per sq ft	5	2.5
Filtered water output	m g per month	240	135
Washwater used	m g per month	8.5	3.6
Washwater used	%	3.5	2.45
Washwater rate, basic	in per min	18	20.5
Washwater rate upper layer	in per min	16	0
Wash duration, basic	min	7	4.5
Wash duration, upper layer	min	3	0
Washes per month	No.	34	29
Average filter run	hrs	21	24-50
Comparative efficiency	%	177.2	100
Initial head loss	ft	2.6	1.6
Final head loss	ft	7.8	7.5

V. J. CALISE,[11] and W. A. HOMER.[12]—The writers are grateful to Mr. K. J. Ives for his pertinent comments on the Russian "biflow" or AKX filter, in which the draw-off pipe for filtered water is located in the upper layer of sand, and raw water enters the filter from the gravel base and from the sand surface with "biflow" established downwards through the top few inches of sand and upwards through the bulk of the graded bed.

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[11] General Sales Mgr., Graver Water Conditioning Co., New York, N. Y.

[12] Senior Process Engr., Graver Water Conditioning Co., New York, N. Y.

While the writers knew of this application, they were not aware of the apparent frequency of its use. The data shown in Tables 5 and 6 indicate quite favorable results comparable to accepted normal filter-clarifier practice. However, nothing is mentioned of the long-range experience with clogging and collection with the internal collector design used. In addition, the problem of possible bypass of only partially-treated, partially-filtered water to the submerged internal collector is a point to be related to American sanitary practice.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3230

OXYGEN BALANCE OF AN ESTUARY

By Donald J. O'Connor,[1] M. ASCE

With Discussion by Messrs. Robert V. Thomann; M. B. McPherson;
C. H. J. Hull; and Donald J. O'Connor

SYNOPSIS

The net seaward movement of organic impurities in estuaries is due to the displacement by the land runoff and to the longitudinal diffusion of the tidal action. The dissolved-oxygen profile depends on the concentration of the organic material, its rate of oxidation, and the resulting rate of reaeration. The interrelationship among these geophysical and biochemical factors is described by a differential equation under a steady-state condition. The assumption of constant coefficients in the equation is confirmed by the field data from estuarine surveys of the Delaware and the James Rivers. Dissolved-oxygen profiles are calculated from the integrated equation and compared to the measured dissolved-oxygen concentrations. The turbulent diffusion coefficient and the deoxygenation rate are determined from an analysis of survey data and are used in the calculation of the oxygen profiles. The agreement between the calculated profiles and the observed values is sufficiently close to justify the use of the equations for the conditions assumed in the development.

INTRODUCTION

The forces of natural self-purification can bring about the return of polluted waters, in time, to a condition of acceptable cleanliness and ultimately to one of natural purity. The time factor is of paramount importance in this process. In a flowing stream, the predominant motion is due to gravity and the formu-

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Note.—Published essentially as printed here, in May, 1960, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2472. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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lations describing this motion are sufficiently accurate to permit calculation of the time of travel. If more accurate determinations are required, reliable and simple methods of measuring velocities in the field are available. The time of travel is markedly changed, however, in those sections of the river which are subjected to tides. The motion of these waters is caused not only by gravity but also by tidal action, density currents, and wind effects. Waste materials which are discharged into estuaries are mixed with the water and are gradually diminished in concentration by the tidal motion, which carries various portions of the pollutant back and forth over many cycles. Any slug of pollutant is ultimately translated to the open sea and this time is referred to as the flushing time rather than the travel time. The flushing time is the resultant of the translating velocity of the river flow and the longitudinal mixing of the tidal action. This concept is of fundamental importance in defining pollution and self-purification of tidal waters. It is the purpose of this work to present a theoretical development defining the processes which determine the distribution of dissolved oxygen in a non-stratified estuary.

REVIEW OF PREVIOUS WORK

In the past ten years (as of 1960), there has been a considerable increase in the study of estuarine mixing and flushing by oceanographers. Although the state of knowledge of these phenomena is far from complete, it has progressed to the point where many useful concepts are presently available to the engineer. An excellent review of estuarine hydrography has been presented by D. W. Pritchard.[2] Estuaries were defined and classified in terms of fresh-water inflow and evaporation, geomorphological structure, and circulation pattern. Pritchard defined an estuary as being "a semi-enclosed coastal body of water having a free connection with the open sea and containing a measurable quantity of sea salt."

Some of the earliest engineering studies of the flushing of pollutants from estuaries have employed the concept of the tidal prism.[3] The assumption was made that complete mixing of the flood waters and polluted harbor waters occurred. This approach was improved by J. Tully[4] and R. G. Tyler[5] who took into account the portions of pollutant remaining after each tidal cycle. B. H. Ketchum further developed[6] these concepts by dividing the estuary into segments, defined by an effective mixing length which is equal to the mean distance covered by the flood tide. Within each segment, mixing is assumed to be complete at high tide and the proportion of water removed and its associated pollution on the ebb tide is determined by the ratio of the intertidal and high tide volumes of the segment. Ketchum stated that incomplete vertical mixing can be taken into account in this hypothesis. A. B. Arons and H. Stommel presented[7] a more mathematical approach, based on a mixing length theory which included considerations of continuity and turbulent diffusion. Stommel in a later work[8] discussed objections to making a priori supposition about the turbulent mixing process. He suggested using the distribution of salinity and river water in order to determine coefficients of turbulent diffusion and to apply these coefficients in analysis of the dilution and distribution of pollutants. Pritchard in a study of the flushing of the Delaware model presented[9] a similar equation for the time rate of change of mean concentration of a conservative substance. This equation was applied to the analysis of the distribution of dye which was injected at various points along the estuarine model. R. E. Kent,[10] following the work of Stommel and Pritchard, evaluated the turbulent diffusion coefficients and successfully applied these to the dye distribution of the model data.

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[2] "Estuarine Hydrography," by D. W. Pritchard, Advances in Geophysics, Academic Press, New York, 1952.

[3] "Pollution of New York Harbor," by E. B. Phelps and C. J. Velz, Sewage Works Journal, Vol. 5, January, 1933.

[4] "Oceanography and Prediction of Pulpmill Polution in Alberni Inlet," by J. Tully, Bulletin No. 83, Fish Reserve Bd., Canada, 1949.

[5] "Disposal of Sewage into Tidal Waters," by R. G. Tyler, Sewage and Industrial Wastes Journal, Vol. 22, May, 1950.

[6] "The Flushing of Tidal Estuaries," by B. H. Ketchum, Sewage and Industrial Wastes Journal, Vol. 23, February, 1951.

[7] "A Mixing-Length Theory of Tidal Flushing," by A. B. Arons and H. Stommel, Transactions, Amer. Geophysical Union, Vol. 32, 1951, p. 419.

The processes of self-purification are similar in tidal waters as in river waters, but certain modifications are necessary to take into account the effect of the high salt concentrations. The progress of the biochemical oxygen demand is retarded when the concentrations of the sea water is greater than 25%. The stage of nitrification is also retarded.[11] The reduced rate of the BOD reaction may also be attributed to the fact that the flushing phenomenon permits portions of organic pollutants to remain within the estuary for extensive periods of time. Thus, a sample of an organic pollutant is made up of portions of various ages, some of which are probably in the nitrification stage. Variation in the most probable number of coliform organisms with tidal cycles has been observed[12] and A. N. Diachishin has defined this variation mathematically with some success for the New York Harbor samples.[13] It is probable that other characteristics are also subjected to this variation. The saturation of dissolved oxygen is less in sea water than in distilled water. The most recent study in this regard is that of G. A. Truesdale, A. L. Downing and G. F. Lowden.[14] Values of the reaeration coefficient are also reduced because of the effect of chloride concentration of the diffusivity of oxygen. The influence is partially offset by virtue of the slight increase in surface tension due to the presence of the inorganic salts. The basic relationships defining the processes of deoxygenation and reaeration in rivers must be related to the flushing times in order to define the oxygen balance in an estuary.

THEORETICAL DEVELOPMENT

An estuary is defined as being a semi-enclosed coastal body of water which is subject to tidal action and in which the sea water, if present, is measurably diluted by river flow. In terms of the geomorphological structure, this study considers the coastal plain estuaries, which characterizes most of the eastern shore of North America. This type has been formed by the drowning of former river valleys, either from subsidence of the land or from a rise in the sea level. Furthermore, non-stratification in both the vertical and lateral directions is assumed to exist.

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[8] "Computation of Pollution in a Vertically Mixed Estuary," by H. Stommel, Sewage and Industrial Wastes Journal, Vol. 25, September, 1953.

[9] "A Study of Flushing in the Delaware Model," by D. W. Pritchard, Tech. Report VII, The Chesapeake Bay Inst., Johns Hopkins Univ., April, 1954.

[10] "Turbulent Diffusion in a Sectionally Homogenous Estuary," by R. E. Kent, Tech. Report XVI, The Chesapeake Bay Inst., Johns Hopkins Univ., April, 1958.

[11] "Effect of Temperature on Biochemical Oxidation of Sewage," by H. B. Gotaas, Vol. XX, May, 1948.

[12] "Sampling for Effective Evaluation of Stream Pollution," by C. J. Velz, Sewage and Industrial Wastes Journal, Vol. 22, May, 1950.

[13] "The Analysis of Water Samples for Cyclical Variations," by A. N. Diachishin, Transactions, ASCE, Vol. 122, 1957.

[14] "The Solubility of Oxygen in Pure Water and Sea Water," by G. A. Truesdale, A. L. Downing and G. F. Lowden, Journal of Applied Chemistry, Vol. 5, 1955, p. 502.

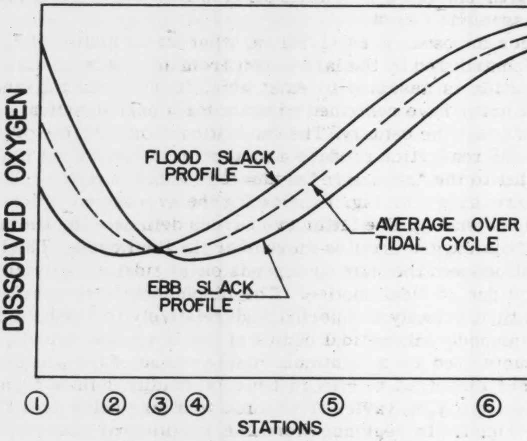
Consider an estuary receiving no other waste matter along its course except that contributed by the land runoff from an upstream discharge. A steady state condition is assumed to exist which implies that the land runoff and the waste discharge have remained constant for a period equivalent to the time of passage through the estuary. The deaeration caused by the organic matter and the resulting reaeration produce a pattern in the dissolved oxygen concentration similar to the "sag curve" evidenced in upland rivers. Dissolved-oxygen profiles are shown in Fig. 1 indicating the average condition and those at the flood and ebb slacks. The latter two curves delineate the limits between which the dissolved-oxygen profiles more over the tidal cycle. The longitudinal displacement between these limits depends on the tidal velocity and period and the turbulence due to tidal motion. The displacement is a maximum for large values of tidal velocity and period and relatively low order turbulence. Conversely, intensely mixed tidal bodies of relatively low velocity and short period are characterized by a minimum displacement of the profiles. The cyclical variation of dissolved oxygen in time is readily defined from these profiles. The dissolved oxygen varies from a low to a high value over the tidal cycle as shown in Fig. 1. In regions where the longitudinal concentration gradient is great, there is a great variation in the dissolved oxygen over the cycle, as at stations 1 and 5. Where the concentration gradient is zero or approaches zero, the variation over the cycle is low, as at stations 3 and 6. Where the concentration gradient is negative, the dissolved oxygen varies inversely as stage height, as at station 1 and where positive, as at station 5, the variation is direct. Inspection of Fig. 1 indicates that the dissolved-oxygen concentration remains substantially constant in time at the flood and ebb slack periods. The profiles therefore can be more readily defined and more accurately measured at these periods than any other. Consider the estuary shown in Fig. 2 and described by the characteristics enumerated above. The x-axis is that of the longitudinal axis of the channel, increasing in the seaward direction. The vertical and lateral axes are designated "y" and "z." The steady state is most suitably defined by either that of the ebb or flood slack, but, from the practical viewpoint, it may also be defined by the average value over the tidal cycle.

The total amount of oxygen carried into the element as shown in Fig. 2 is that contributed by (a) the flowing stream across the plane 1, (b) turbulent transport across the plane 1, and (c) turbulent transport across the plane 2.

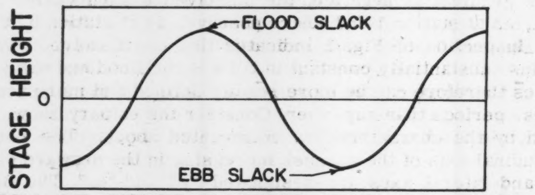
The total amount of oxygen carried out of the element is affected by the same factors through the planes 3 and 4, plus that removed by the oxidation of the organic matter. The concentration gradients at each plane of the element in both the x and y are also shown in Fig. 2. Utilizing the concentrations and the associated gradients in a material balance in and out of the element leads to the partial differential equation which defines the oxygen distribution under a steady state:

$$0 = \epsilon_x \frac{\partial^2 c}{\partial x^2} + \epsilon_y \frac{\partial^2 c}{\partial y^2} - U \frac{\partial c}{\partial x} - K_d L \ldots\ldots\ldots\ldots (1)$$

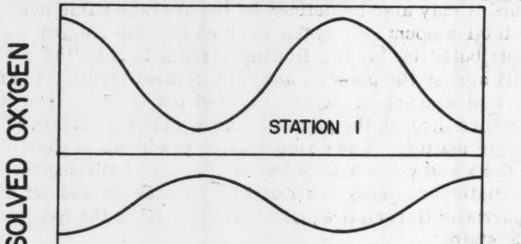
in which ϵ_x and ϵ_y are the turbulent transport coefficients in the x and y direc-



(*a*) VARIATIONS OF DO IN DISTANCE UNDER STEADY STATE



(*b*) VARIATION OF STAGE IN TIME DUE TO TIDES



(*c*) VARIATION OF DO IN TIME DUE TO TIDES

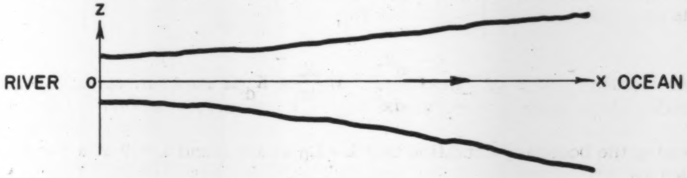
FIG. 1.—DO VARIATION DUE TO TIDES

tions, c is the concentration of oxygen, L denotes the ultimate biochemical oxygen demand, U is the forward velocity of the river flow, and K_d represents the coefficient of deoxygenation.

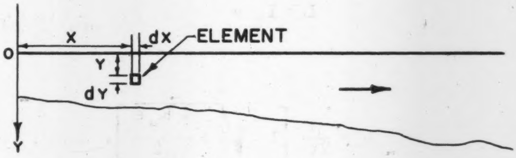
In a non-stratified estuary, the gradient in the vertical plane is essentially zero and the amount of oxygen carried into the element has been shown to be controlled by that passing through the water surface.[15] This condition reduces the partial differential to an ordinary differential as follows:

$$0 = \epsilon_x \frac{d^2 c}{dx^2} - U \frac{dc}{dx} + K_2 \left[c_s - c \right] - K_d L \quad \ldots\ldots\ldots\ldots (2)$$

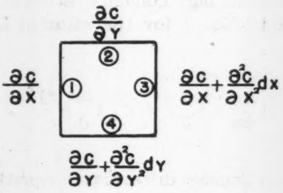
in which c_s is the saturation value of oxygen at the given temperature, and K_2 denotes the reaeration coefficient.


Z
RIVER
O
X OCEAN

(*a*) PLAN


X
dX
ELEMENT
O
Y
dY
Y

(*b*) SECTION


$\frac{\partial c}{\partial Y}$
②
$\frac{\partial c}{\partial x}$
①
③
$\frac{\partial c}{\partial x} + \frac{\partial^2 c}{\partial x^2} dx$
④
$\frac{\partial c}{\partial Y} + \frac{\partial^2 c}{\partial Y^2} dY$

(*c*) ELEMENT CONCENTRATION GRADIENTS

FIG. 2.—ESTUARINE CONCENTRATION GRADIENTS

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[15] "Mechanism of Reaeration in Natural Streams," by D. J. O'Connor and W. E. Dobbins, Transactions, ASCE, Vol. 123, 1958, p. 641.

In Eq. 2 the first term represents the rate of turbulent flux, produced by the tidal action, the second term is that due to the river discharge, and the third and fourth terms are, respectively, the reaeration and deoxygenation rates. The form of Eq. 2 is based on a constant value of the eddy diffusivity and a uniform channel cross section. In any actual case, neither of these conditions hold, but average values may be used if the variations from the actual are within certain statistical limits. The magnitude of the error introduced by these approximations may be evaluated in any particular case. Furthermore, Eq. 2 is developed on the basis of an increasing oxygen gradient with distance; that is, that portion of the sag curve downstream from the location of the minimum dissolved oxygen concentration. The final solution of the equation is the same if the differential equation is presented in terms of a decreasing dissolved gradient which is upstream from the minimum point. Before Eq. 2 can be integrated the variable L must be replaced as a function of x. This may be done by a similar material balance as performed for the oxygen concentration and leads to

$$0 = \epsilon \frac{d^2L}{dx^2} - U \frac{dL}{dx} - K_d L \ldots\ldots\ldots\ldots\ldots (3)$$

Imposing the boundary condition that $L = L_0$ at $x = 0$ and $L = 0$ at $x = \infty$ Eq. 3 is solved as

$$L = L_0 e^{j_1 x} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which

$$j_1 = \frac{U}{2\epsilon} \left[1 - \sqrt{1 + \frac{4 K_d \epsilon}{U^2}}\right] \ldots\ldots\ldots\ldots\ldots (5)$$

Eq. 4 is similar to that developed[16] by H. R. Thomas, Jr. and R. S. Archibald to define longitudinal mixing through conduits, streams and tanks.

Substitution of Eq. 4 into Eq. 2 for the value of L and rearranging terms there results

$$\epsilon \frac{d^2c}{dx^2} - U \frac{dc}{dx} - K_2 c = K_d L_0 e^{j_1 x} - K_2 c_s \ldots\ldots\ldots\ldots (6)$$

Eq. 6 is a second-order linear differential equation with constant coefficients and may be integrated by standard procedures. Subject to the boundary conditions that $c = c_0$ at $x = 0$ and $c = c_s$ at $x = \infty$, this equation leads to:

$$c = c_0 e^{j_2 x} + c_s \left[1 - e^{j_2 x}\right] - F L_0 \left[e^{j_1 x} - e^{j_2 x}\right] \ldots\ldots (7)$$

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[16] "Longitudinal Mixing Measured by Radioactive Tracers," by H. A. Thomas, Jr. and R. S. Archibald, Transactions, ASCE, Vol. 117, 1952, p. 839.

Eq. 7 may be expressed in terms of the dissolved-oxygen deficit as follows:

$$D = F\,L_0\left[e^{j_1 x} - e^{j_2 x}\right] + D_0\,e^{j_2 x} \quad \ldots\ldots\ldots\ldots (8)$$

in which

$$F = \frac{K_d}{K_2 + U_{j_1} - \epsilon\, j_1^2} = \frac{K_d}{K_2 - K_d} \quad \ldots\ldots\ldots\ldots (9)$$

and

$$j_2 = \frac{U}{2\,\epsilon}\left[1 - \sqrt{1 + \frac{4\,K_d\,\epsilon}{U^2}}\,\right] \quad \ldots\ldots\ldots\ldots (10)$$

The similarity between Eq. 8 and that developed[17] by H. W. Streeter and E. B. Phelps for rivers is evident. A unique condition can result in the following equality:

$$K = \frac{2\,U^2}{\epsilon} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

In this case Eq. 5 reduces to

$$j = -\frac{K}{2\,U} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12a)$$

or

$$jx = -\frac{K\,t}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (12b)$$

This condition, in turn, reduces Eq. 4 to one very comparable to that characteristic of upland rivers.

If the land runoff is very low or the cross-sectional areas large, which result in insignificant values of velocity, then Eq. 8 becomes

$$D = \frac{K_d\,L_0}{K_2 - K_d}\left[e^{-x\left(\frac{K_d}{\epsilon}\right)^{1/2}} - e^{-x\left(\frac{K_2}{\epsilon}\right)^{1/2}}\right] + D_0\,e^{-x\left(\frac{K_2}{\epsilon}\right)^{1/2}} \quad \ldots (13)$$

Before Eqs. 8 or 13 can be used in a practical case, the velocity term, the eddy diffusivity and the coefficients of reaeration and deaeration must be assigned numerical values.

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[17] "A Study of the Pollution and Natural Purification of the Ohio River," by H. W. Streeter and E. B. Phelps, Public Health Bulletin 146, USPHS, Washington, D. C. 1925.

For those cases, in which the coefficients of reaeration and deaeration are of the same order and the initial deficit is very low, then the sag equation of Streeter and Phelps may be used as an approximation for estuaries. This approximation is valid because the term in parenthesis in Eqs. 8 or 13 is a difference of exponential terms. Consequently, if the exponents are taken as K_d t and K_2 t, as in the original sag equation, then the deficit is approximately equal to that obtained by using the exponents indicated for Eqs. 8 or 13. When the initial dissolved oxygen deficit and the difference between the coefficients of reaeration and deaeration are significant, then Eqs. 8 or 13 must be used.

EVALUATION OF THE COEFFICIENTS

The turbulent transport coefficient may be determined from a knowledge of the concentration distribution of any property in the estuary. The chloride concentration or the salinity is taken as the required steady state property. The source of this property is the coastal waters of the ocean. The turbulent flux, caused by the tidal action, diffuses salt waters into the mouth of the river. The distribution and extent of this characteristic reflects the magnitude of the eddy transport coefficient. The steady state condition predicates fresh water from one major source, with none added by ground water and removed by evaporation. The material balance of this property may be constructed in a fashion similar to that of the oxygen-demand material, which leads to

$$0 = \epsilon \frac{d^2 s}{dx^2} - U \frac{ds}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

In Eq. 14, s refers to the concentration of salinity or chloridity and the remaining terms have been previously defined. Since this is a conservative property, no decay term is included. The eddy diffusivity and the velocity are assumed to remain constant over the length of the estuary considered. Both Stommel[8] and Kent[10] more correctly assumed these parameters to vary with distance and evaluated the eddy coefficient by expressing Eq. 14 in the finite-difference form:

$$\epsilon = \frac{2\,\Delta x\, s_x U}{s_{x+\Delta x} - s_{x-\Delta x}} \quad \ldots\ldots\ldots\ldots\ldots \quad (15)$$

In Eq. 15 2 Δx delineates a finite section of estuary over which the salinity may be assumed to vary linearly. The mean concentration over this section is s_x and the mean velocity U. The demoninator is the difference between the concentrations at the limits of the section. For the type of estuary considered, the concentration at the seaward end is greater than at the river end and ϵ is always positive. At both the head and the mouth of the estuary the gradient approaches zero and neither Eq. 14 nor 15 is appropriate. In this work, the assumption is made that the diffusivity and velocity do not vary with distance over the section of the estuary considered. The error introduced by this approximation may be determined by comparing the value of the eddy diffusivity as calculated by Eq. 15 and by the integrated form of Eq. 14. Assuming constant coefficients in this equation, integration leads to:

$$s = s_0 e^{-\frac{U}{\epsilon}x} \qquad (16)$$

in which s is the salinity at distance x, and s_0 is the salinity at x = 0. A plot of the logarithm of the saline or chloride concentration against an arithmetic distance scale will indicate a linear relation if the coefficients U and ϵ do not vary with distance. The slope of such a line is a measure of the ratio of these coefficients and ϵ may be evaluated graphically in this fashion. In some cases, the section of river under study may be above the intrusion of sea water and salinity data would not be available for evaluation of the turbulent transfer coefficient. In this case, any conservative property of the waste water, which significantly increases the river-water concentration, may be used to determine the eddy coefficient. A particularly appropriate characteristic is the concentration of a stable dissolved chemical ion. The solution proposed by Stommel[8] is appropriate for this case:

$$\frac{s}{s_o} = \frac{1 - e^{(U/\epsilon)x}}{1 - e^{(U/\epsilon)x_0}} \qquad (17)$$

in which s_o is the upstream concentration of the conservative property, s denotes the downstream concentration of the conservative property, x_o is the distance from the mouth of the estuary or from the point where the concentration becomes constant to the point where concentration = s_o, and x represents the distance from the mouth of the estuary or from the point where the concentration becomes constant to the point where concentration = s, the distance measured negatively from the mouth or from the point of constant concentration.

The flow velocity, U, may be calculated directly from the continuity equation

$$U = \frac{Q}{A} \qquad (18)$$

in which Q is the river discharge and A is the cross-sectional area. It is probable that the velocity will also vary with distance, decreasing in the downstream direction. The error introduced by assuming an average value may be determined, as in the case of the eddy diffusivity. Frequently, the value of this velocity is so small that no significant error is introduced in the oxygen calculations by this assumption. In any case, the cross-sectional area is frequently composed of two distinct areas: the relatively shallow lateral zone and the deep central channel zone. The quantity of flow in this latter zone is usually so much greater than in the lateral area that it may be appropriate in determining the velocity by means of Eq. 18 to employ only the central channel area. Each estuary must be evaluated separately in this regard and examination of the shape of the cross-sectional areas indicates the possibility of this channeling effect.

The reaeration coefficient may be determined from formulas which were developed specifically for non-tidal streams and rivers.[15] The mathematical model proposed for the non-tidal bodies may be applied to estuaries. The basic relationship for the reaeration coefficient is

$$K_2 = \frac{\left[D_L\, r\right]^{1/2}}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

in which D_L is the molecular diffusivity of oxygen, r denotes the rate of surface renewal, and H is the average depth of the section. The surface renewal rate was defined by the ratio of the time averages of the velocity fluctuation and mixing length in the vertical plane. This ratio is equal to the velocity gradient at the water surface for a condition of non-isotropic turbulence. This parameter probably defines the surface renewal for a vertically stratified estuary, in which there are pronounced differences in the velocity with depth. For a non-stratified estuary, however, the velocity field is reasonably uniform in the vertical plane. For this case, the vertical velocity fluctuation and the mixing length were taken to be equal to one-tenth of the forward flow velocity and average depth. These values were reported as averages from measurements taken in tidal bodies.[18,19] It is significant to note that these ratios are of the same order of magnitude as in non-tidal rivers. Substituting the ratio of the average velocity and average depth for the rate of surface renewal, the reaeration coefficient becomes:

$$K_2 = \frac{\left[D_L\, U_o\right]^{1/2}}{H^{3/2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

The velocity U_o refers to mean tidal velocity over a complete cycle. This velocity is usually more significant than that due to river discharge. The depth refers to the average over the section and over a cycle. If both the tidal velocity and the depth vary considerably with distance, then short stretches must be taken over in which the variation is minimal.

The evaluation of the rate of deoxygenation may be carried out in a manner similar to that of the eddy diffusivity. Assuming representative values of the eddy diffusivity and the velocity are available, the value of the deoxygenation coefficient, K_d, may be obtained from Eq. 4. The logarithm of the concentration of organic matter (BOD) is linear with distance for the stated conditions. The slope of the line defining this relation is a measure of the exponent in Eq. 4, from which K_d may be calculated. In assigning a value of K_d to represent the decay of organic matter, distinction should be made between the removal of organic matter and the rate of deoxygenation. In the above equation, it has been assumed that the rate of removal of organic matter is equal to the rate of deoxygenation. If BOD is being removed by other mechanisms, such as sedimentation, which do not utilize oxygen, then the coefficients associated with BOD must take this difference into account. In such a case, the designation of K in Eqs. 3 and 4 becomes K_r to indicate total removal of BOD. In Eq. 8 and the subsequent equations which describe the oxygen balance, the K_d is appropriate. If no other processes are involved in the removal of BOD, the K_d will

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[18] "Theoretical Considerations of the Motion of Salt and Fresh Water," by J. B. Schiff and J. T. Schonfeld, Proceedings, Minn. Internatl. Hydr. Convention, September, 1953.

[19] "Density Current Problems in an Estuary," by T. Hamada, Proceedings, Minn. Internatl. Hydr. Convention, September, 1953.

equal K_r. These factors have been described[20] and examples of their application presented.[21]

COMPARISON OF THE CALCULATED AND OBSERVED PROFILES

In order to verify the proposed formula, comparisons are made between the calculated profiles and values measured in estuarine pollution surveys. These comparisons may be made on both the BOD decay (Eq. 4) and the DO balance (Eq. 8). These equations are based on steady state conditions and for a valid comparison, the observed data should be representative of such a condition. Sampling should be performed simultaneously at a series of stations along the estuary when the variations of tidal motion, flow, temperature and waste discharge are minimal. This steady state condition is maintained at high or low water slack, as shown in Fig. 1, for a period of 1 hr to 3 hr depending on the type of tide. Appreciating these factors, A. J. Kaplovsky conducted and reported estuarine surveys of the lower Delaware River using the "same slack" technique.[22,23] Prior to these surveys, Kaplovsky had conducted intensive sampling at various stations over the width and the depth of the cross section. Sampling was carried on at low and high water slack periods. Inspection of these data indicate no definite pattern of variation of DO and BOD in the lateral and vertical planes. The variation of the BOD within each set at any slack period was greater than that of the DO. The variation of the latter was minimal. Kaplovsky compared the data from the cross-sectional series and the "same slack" series and concluded that the latter were more representative of the actual conditions. Six surveys were conducted and of these three provided the complete information for a comparison between the calculated DO profile and the observed values.

Prior to the time the Delaware River work was reported, the Water Control Board of the State of Virginia conducted surveys on the Lower James River, which is subject to the tidal action of the Chesapeake Bay. Eleven surveys were run during the months of August and September.[24] Although the data are not as reliable a measure as those from a survey of a "same-slack" period, the average results do provide a sufficiently representative pattern to justify a comparison between the calculated DO profile and the observed results. Intensive cross-sectional sampling was also conducted.

In order to calculate a dissolved oxygen profile, knowledge of the various coefficients, representing the velocity term, the eddy diffusivity and the coefficients of deoxygenation and reaeration, is necessary.

Calculation of the Velocity Term.—The value of the velocity used in the oxygen-balance equation is the seaward velocity due to the river discharge. In the case of the Delaware, the cross-sectional area used in this calculation was that of the ship channel. Based on Pritchard's observations, Kaplovsky estimated that the channel, which occupied approximately 70% of the cross-sectional area, carried more than 90% of the waste. The river discharge at a specific station was determined by multiplying the flow at Trenton, N. J., by the ratio of the drainage area at that station to that at Trenton.

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[20] "Pollution Load Capacity of Streams," by H. A. Thomas, Jr., Water and Sewage Works, Vol. 95, November, 1948.

[21] "Assimilative Capacity of Natural Rivers," by D. J. O'Connor, Proceedings, Manufacturing Chemists Assn., 1959 (in press).

[22] "Investigation of Sanitary Water Quality in the Lower Delaware River," by A. J. Kaplovsky, Tech. Report II, Water Pollution Comm., State of Del., December, 1956.

[23] "Estuarine Pollution Surveys," by A. J. Kaplovsky, Sewage and Industrial Wastes Journal, Vol. 29, September, 1957.

[24] "Lower James River Pollution Study," Water Control Bd., Commonwealth of Va., 1951.

The cross sections of the James River are characterized by littoral areas of about 4 ft in depth and a main channel of approximately 20 ft in depth. The majority of the flow was restricted to the latter zone, similar to that of the Delaware. The area of the main channel and adjacent littoral area up to 4 ft in depth were taken as the flow area. As in the case of the Delaware River, the flow area was approximately 70% of the total cross-sectional area. The velocity term was determined by means of Eq. 18. The river discharge, Q, was taken as the average value of a period of about 2 weeks preceding the survey date.

Evaluation of the Eddy Diffusivity.—In the mathematical development, it was assumed that the coefficient of eddy diffusion remained constant over the section of the estuary under consideration. If this assumption is valid, then a plot of the logarithm of the chloride concentration with an arithmetic scale of distance yields a straight line. For the Delaware River, the approximate linearity of this relationship is indicated in Fig. 3. The line is fit by eye and the coefficient of eddy diffusion is calculated in accordance with Eq. 16.

In the case of the James River, the salt water intrusion did not extend into the zone of the DO sag curve. Consequently, the chloride concentrations could not be used in the determination of the eddy diffusion coefficient. It was fortuitous that one of the industrial discharges contain a significant quantity of sulfate concentration and the measured values of sulfate ion were used in the determination of the diffusion coefficient, by means of Eq. 17. Analysis of the sulfate data indicated x_0 may be taken as 20 miles.

Assignment of the Coefficient of Deoxygenation.—This coefficient may be determined in a manner similar to that of the eddy diffusivity. Having calculated the velocity and eddy diffusivity, as indicated previously, and having available measured values of BOD at various estuarine stations, the value of the deoxygenation coefficient may be obtained from Eqs. 4 and 5. The logarithm of BOD is plotted against an arithmetic scale of distance and a line of best fit approximated by eye. The slope of the line is a measure of the exponent in Eq. 4 and the coefficient of deoxygenation is calculated from Eq. 5. This relationship is shown in Fig. 3 for the Delaware River and Fig. 4 for the James River. The river-water temperatures are also indicated in these figures. In calculating this coefficient for other sets of data at different temperatures, the above procedure was followed and, in addition, the following temperature correlation was used:

$$K_T = K_{20} \, (1.047)^{T-20} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

in which K_{20} is the coefficient at 20°C, K_T denotes the coefficient at T°C, and T is the temperature, in degrees Centigrade. Each set of data is consistent with respect to temperature within the group of each river. Because of the erratic nature of the decrease in BOD, this coefficient proved to be the most difficult to define. Trial-and-error computations were necessary in order to achieve this consistency.

Determination of the Reaeration Coefficient.—From extensive measurements of cross-sectional areas of both rivers by the Corps of Engineers, average depths were calculated. The value used in Eq. 20 was the average of the flood and ebb conditions. The mean tidal velocities over a cycle were taken from

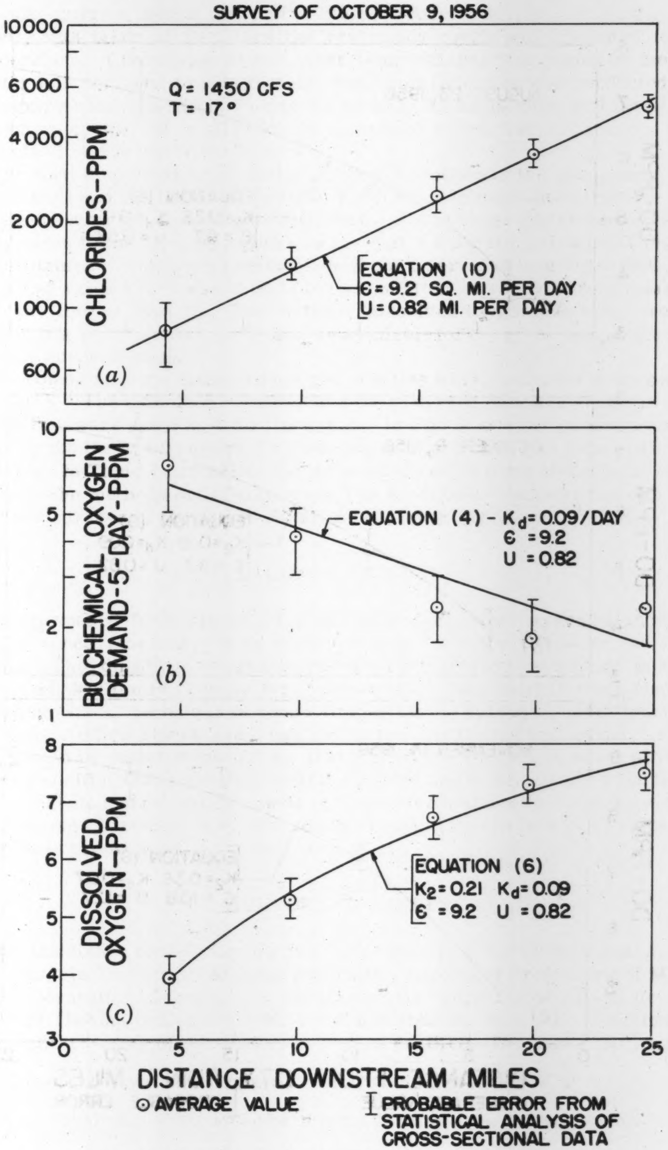

FIG. 3.—CHLORIDE, BOD, AND DO PROFILES FOR THE DELAWARE RIVER

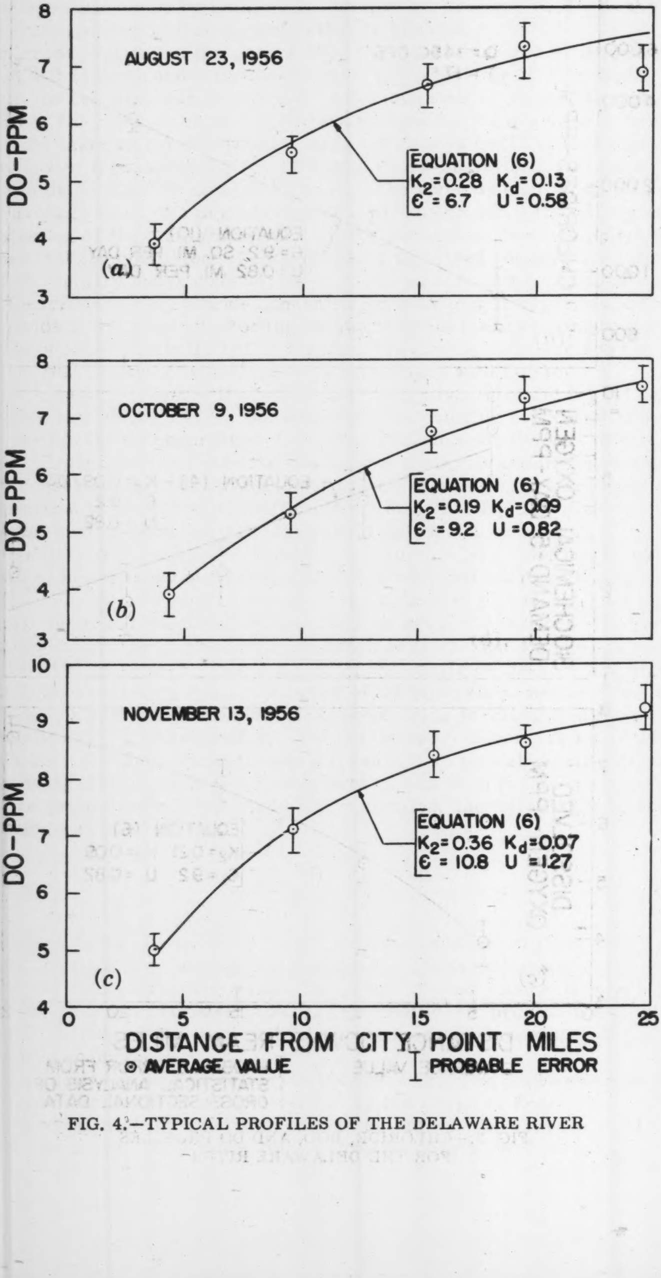

FIG. 4.—TYPICAL PROFILES OF THE DELAWARE RIVER

the tidal current data of the United States Geodetic Survey. The oxygen diffusivity was taken at 20°C and the reaeration coefficient computed for this temperature. Conversion to the other temperatures was made by means of Eq. 21. It is pertinent to note that one survey of Delaware was conducted while an average wind velocity of 15 mph prevailed. An adjustment was made in the reaeration coefficient to allow for the increased oxygen transfer induced by the wind effect on the water surface.[25]

The final step before calculation of the DO profiles is the assignment of ultimate BOD and the initial DO deficit. The former is calculated from the 5-day values, knowing the laboratory coefficient. The average factor converting the 5-day value to the ultimate value was taken at 1.3 for the Delaware[26] and 1.67 for the James[27] which were determined from a series of long-term BOD tests. The 5-day value which was used in the calculation of the DO profile was read from the line of best fit. The initially dissolved oxygen was taken from the survey data and the saturation value is available for the given temperature and chloride concentration.

With these data, the dissolved-oxygen profiles were computed in accordance with Eq. 8. The calculated profiles and the observed data are shown in Fig. 3 for the Delaware and Fig. 4 for the James, In Fig. 5, similar comparisons are made for the three surveys of the Delaware in which complete information was available. In Table 1 are shown the associated coefficients and data, as determined by the methods outlined in steps 1 to 4. Similar comparisons are made for three surveys of the James River, as shown in Fig. 6.

CONCLUSIONS

The proposed formulas defining the BOD decay and oxygen balance in an estuary agree reasonably with observed data from the Delaware and James Rivers. The dissolved-oxygen profiles are more consistent and in better agreement than the BOD decay for these rivers. This work further confirms the "same-slack" technique of conducting estuarine surveys. Additional survey data of other rivers are required to test the theoretical model. Future work should be directed toward an analysis of the conditions which allow for variable coefficients in the differential equation and a non-steady state toward more accurate and reliable methods of field determinations and measurements. The proposed formulas may also apply to upland rivers in which longitudinal diffusion is pronounced.

ACKNOWLEDGMENTS

The assistance of the following men is gratefully acknowledged and appreciated: Francis Taylor and Anthony Ventriglia, Associate Professors of Mathematics, Manhattan College, who checked some of the mathematical development; and Thomas Gallagher, A.M. ASCE, and John St. John, who compiled data and calculated many of the dissolved-oxygen profiles. The participation of the

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[25] "Effects of Winds on the Oxygen Transfer Coefficient," by D. J. O'Connor, unpublished manuscript.

[26] A. J. Kaplovsky, Water Pollution Comm., State of Del., private communication, 1958.

[27] A. L. Paessler and D. Aderholt, Water Control Bd., Commonwealth of Va., private communication, 1959.

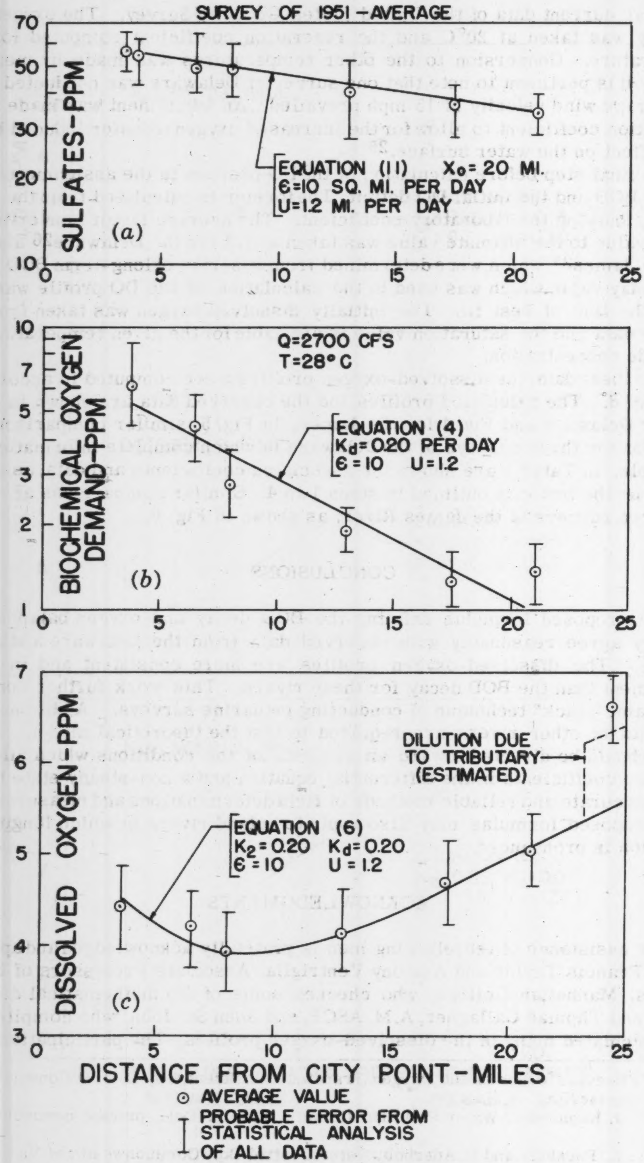

FIG. 5.—SULFATE, BOD, AND DO PROFILES FOR THE JAMES RIVER

TABLE 1.—BASIC DATA USED IN CALCULATION OF ESTUARINE DISSOLVED OXYGEN PROFILES

DELAWARE RIVER

Data of 1956

Survey Tide	August 23 Low Water Slack	October 9 Low Water Slack	November 13 High Water Slack
Q - CFS	5300	7450	11,600
U - Mi. PD	0.58	0.82	1.27
T - Centigrade	25	17	12
Kd - Per Day	0.13	0.09	0.07
K_2 - Per Day	0.28	0.19	0.36
C_s - PPM	8.1	9.4	10.4
Co - PPM	4.1	3.9	5.0
L_5 - PPM	3.5	6.3	9.5
Lo - PPM	4.6	8.2	12.4
- Sq. Mi. PD	6.7	9.2	10.8

JAMES RIVER

Data of 1951

Survey	Average	August 20, 21, 22	August 27, 28, 29
Q - CFS	2700	3150	2350
U - Mi. PD	1.2	1.4	1.05
T - Centigrade	28	30	26
Kd - Per Day	0.20	0.22	0.18
K_2 - Per Day	0.20	0.20	0.17
Cs - PPM	7.7	7.5	8.0
Co - PPM	4.5	5.0	3.4
L_5 - PPM	5.4	6.0	3.6
Lo - PPM	9.0	10.0	6.0
- Sq. Mi. PD	10.0	11.5	9.0

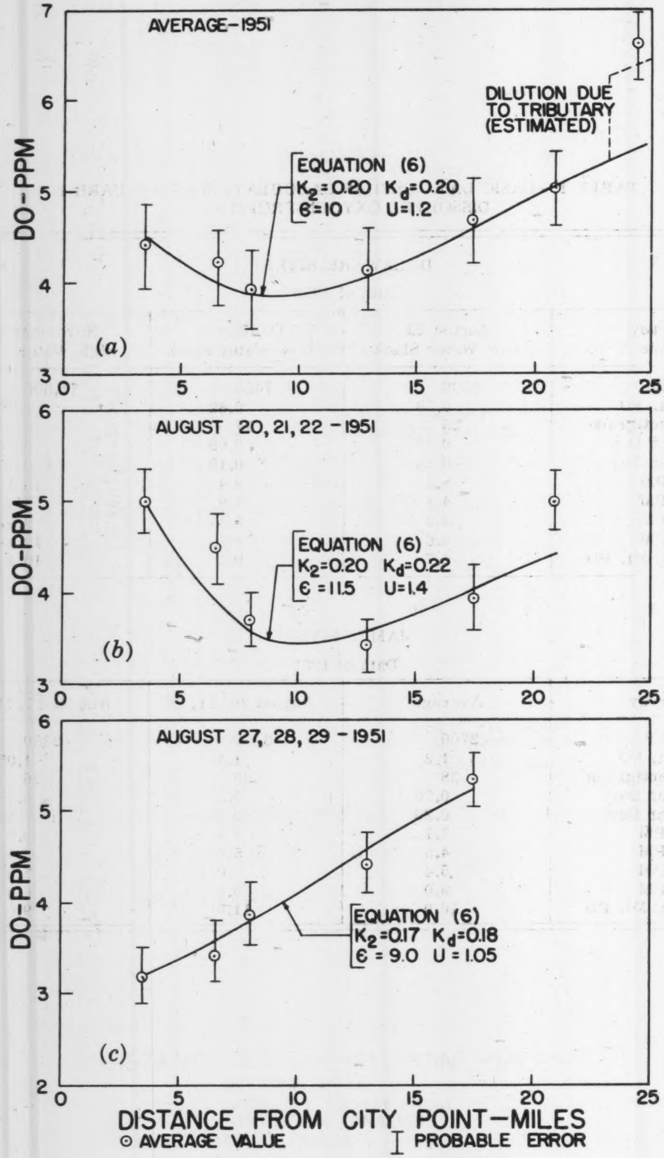

FIG. 6.—TYPICAL DO PROFILES OF THE JAMES RIVER

latter was sponsored by a grant from the National Science Foundation for student participation in research work.

APPENDIX I

The coordinates of the point of minimum dissolved oxygen, usually referred to as the critical point, may be derived as follows: Differentiation of Eq. 8 and equating to zero yields the expression for the distance to the critical point:

$$x_c = \frac{1}{j_1 - j_2} \log_e \frac{j_2}{j_1} \left[1 - \frac{D_0}{L_0 F} \right] \ldots\ldots\ldots\ldots \quad (22)$$

The expression for the minimum deficit is obtained by taking the second derivative of Eq. 8, substituting this derivation in Eq. 6 and permitting dc/dx to equal zero. The minimum deficit equals:

$$D_c = \frac{K_d L_0 e^{j_1 x}}{K_2} \left[1 + \frac{j_1^2 \epsilon}{K_2 - K_d} \right] - \frac{\epsilon j_2^2 e^{j_2 x}}{K_2} \left[F L_0 - D_0 \right] \ldots \quad (23)$$

For the special case in which the coefficients of reaeration and deoxygenation are equal, Eq. 8 becomes:

$$D = \frac{K x L_0}{U - 2 \epsilon j} e^{jx} + D_0 e^{jx} \quad \ldots\ldots\ldots\ldots \quad (24)$$

APPENDIX II.—NOTATION

The following symbols have been adopted for use in this paper:

A = mean cross-sectional area of estuary;

c = concentration of dissolved oxygen:

c_o = initial concentration of dissolved oxygen;

c_s = saturation valve of dissolved oxygen;

D = dissolved oxygen deficit:

D_o = initial dissolved oxygen deficit;

D_c = critical dissolved oxygen deficit;

D_1 = coefficient of molecular diffusion of oxygen;

F = term in Eq. 9;

H = mean depth of estuary;

j_1 = term in Eq. 5;

j_2 = term in Eq. 10;

K_d = coefficient of deoxygenation;

K_r = coefficient of BOD removal;

K_2 = coefficient of reaeration;

L = ultimate BOD:

L_o = ultimate BOD at initial point;

L_5 = 5-day BOD;

Q = land runoff;

r = rate of surface renewal;

s = concentration of a conservative characteristic:

s_0 = initial concentration of a conservative characteristic;

T = temperature, Centigrade;

U = velocity due to land runoff:

U_o = mean tidal velocity;

x = longitudinal distance;

y = vertical distance;

z = lateral distance; and

ϵ = coefficient of eddy diffusion

DISCUSSION

ROBERT V. THOMANN,[28] A. M. ASCE.—The author is to be congratulated for a sincere attempt at describing theoretically the complex mechanisms underlying the dissolved oxygen distribution in an estuary. However, there are several areas in the development which require further clarification.

The development assumes non-stratification in both the lateral and vertical directions. As additional data is being collected in different estuaries along the east coast, the assumption of non-stratification may prove to be a poor one. Extreme caution must be used in assuming non-stratified conditions on the basis of limited data. Indeed, the very definition of "non-stratified" should be made more clear since the author implicitly assumes some degree of lateral stratification by using the "probable error" of the cross section results. This degree of stratification is approximately 0.7 ppm of dissolved oxygen as determined from Fig. 4.

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[28] San. Engr., U. S. Public Health Service, New York Univ., Dept. of Meteorology and Oceanography, New York, N. Y.

There appears to be some confusion with regard to Fig. 1(b) and 1(c). For both the Delaware and James Rivers which are characterized by a progressive wave, the movement of the dissolved oxygen profile during a tidal cycle is determined primarily by the variation in tidal currents. For the two rivers under consideration, flood slack current occurs about 2-3 hr after high water. If Fig. 1(b) and 1(c) are meant to be an illustration of the Delaware and James Rivers, they are somewhat misleading. Fig. 1(c) should show minimum dissolved oxygen at approximately the time the stage height curve is crossing the zero axis.

Mr. O'Connor's approach appears to break down into the following steps:

(a) The determination of a suitable fresh water inflow rate to arrive at the forward velocity, U,

(b) the graphical determination of the eddy diffusion term, ϵ, using the U obtained from (a) and suitable field data,

(c) the graphical determination of the coefficient, j_1,

(d) the solving for K_d using (a), (b), and (c) and Eq. 5,

(e) the determination of the reaeration coefficient, K_2, from suitable field data,

(f) the solving for the coefficient, j_2 using (a), (b), and (e) and Eq. 10,

(g) solving for the quantity F by means of Eq. 9, and

(h) the solution of Eq. 7 using j_1, j_2, F, and field data.

The author fails to make clear the mechanism used to arrive at the forward velocity needed. For the Delaware, it is not clear whether the flow on the same day of sampling was used or whether some lag period was allowed. Recent model studies conducted by the Corps of Engineers at the Delaware model in Vicksburg, Mississippi for the Interstate Commission on the Delaware River and the City of Philadelphia, have indicated that the time of travel of the centroid of mass of a dye release at Trenton, New Jersey to the stretch of the Delaware under consideration might vary from about 10 days for the higher flows to as much as 30 - 90 days for the lower flows. The determination of the rate of fresh water inflow to be used in the application of this approach is critical and an extremely difficult one, the procedure of which should be indicated. As is discussed subsequently, the development allows for considerable latitude in this regard, thereby indicating the possible insensivity of the approach to the magnitude of the fresh water inflow.

Step (b) requires suitable data of a conservative variable existing in the estuary in sufficient quantities to be measured with present techniques. It is fortuitous that the stretch of the Delaware under consideration afforded this variable in terms of the chloride content. However, in the upper end of the Delaware estuary, it is not believed that such a constituent exists.

Regarding the graphical approach used in determining the eddy diffusion term, the basic data for the chloride content measured during the November 13, 1956 series (Fig. 4) was given to another individual unfamiliar with this field but familiar with approximating a straight line to a set of points by eye. This individual obtained an ϵ of 9.2 exactly the same as the author's for the

October 9, 1956 series (Fig. 4). Again, the approach does not appear to be sensitive enough to distinguish the difference between these values.

The graphical determination of the coefficient j_1 is subject to the same errors although in this case the prior mentioned individual did somewhat better estimating a j_1 of about -0.04 as compared with -0.047 computed from the data contained in the paper.

The remaining steps of the development are fairly straightforward assuming one is willing to accept the application of Eq. 20 to estuaries and Eq. 21 to the reaeration coefficient. It would have been helpful if the author had indicated the survey of the Delaware for which an "adjustment" of the K_2 value was made and at least generally, the nature of the adjustment.

It is important to note that suitable field data must be available to determine c_0 and c_s. If one were placed in the position of using this development to predict, for example, the effect of increasing the pollutional load to the stream, these quantities would have to be assumed.

The writer using the data from the November 13, 1956 series duplicated the procedure outlined using, however, the eddy diffusivity of 9.2 indicated above and the U of 0.82 used in the October 9, 1956 series. In view of the transit time from Trenton, New Jersey to the area under consideration, this latter quantity can be reasonably justified. The use of these values yielded a dissolved oxygen profile almost exactly similar to the November 13th series indicated in Fig. 4. Hence, the use of a flow rate of 7,450 cfs instead of 11,600 cfs and a turbulent transport coefficient of 9.2 instead of 10.8 resulted in a similar profile. Both the flow rate of 7,450 cfs and an ϵ of 9.2 could have logically been chosen to determine the final profile. One wonders then, whether the development actually is able to distinguish definitive cause and effect relationships.

M. B. MCPHERSON,[29] **M. ASCE.**—This discussion is addressed to the hydraulic aspects of the paper. In particular, the procedure employed in arriving at a constant diffusion coefficient will be examined and the absence of an adequate theory describing the time-distribution of fresh water inflow will be recognized. Only the Delaware River situation will be considered.

In a recent discussion,[30] various contemporary non-conservative pollution studies of the Delaware River were briefly described. The following paragraph from the discussion is directed to the author's paper:

> "After (Kent's) paper was published, O'Connor compared calculated with observed dissolved oxygen profiles for one HWS (high water slack) and two LWS (low water slack) boat sampling runs made in the latter half of 1956 by the State of Delaware Water Pollution Commission. The profiles used were for the rising end of the curves beyond the minimum concentrations, for five stampling points extending from Station +193 to Station +305. Since the data was collected at the "same slack" and the river flow was "low," it was assumed conditions were steady and diffusion coefficients were calculated from the observed chloride profiles. In addition, it was assumed that the diffusion coefficient, the channel cross section and the mean transport velocity were constant over the reach studied. O'Connor suggests that the distribution of a stable dissolved chem-

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[29] Prof. of Hydr. Engrg., Dept. of Civ. Engrg., Univ. of Illinois, Urbana, Ill.

[30] Discussion of "Diffusion in a Sectionally Homogeneous Estuary," by M. B. McPherson, Proceedings, ASCE, Vol. 86, No. SA 5, September, 1960.

ical ion might be utilized as a spacial tracer for the section of the river upstream from the salinity front. Since there are no significantly distributed spacial tracers extant in most of the non-saline section, a special tracer would have to be introduced prior to sampling. Although O'Connor cites fresh-water inflow rates, they are not necessarily the true rates which contributed to mixing and transport."

Because model data will be introduced, the approximate relations between the stationing used in the paper, by the State of Delaware Water Pollution Commission[22] and by the Waterways Experiment Station, are given in Table 2. The chloride data for the three Delaware River surveys are listed in Table 3 and plotted in Fig. 7. Only one of the three chloride profiles has been graphed in the paper, Fig. 3(a), Survey of October 9, 1956. Data for two of the seven locations occupied in each survey were excluded by the author. Inasmuch as wastes discharged into the upper river contribute chlorides to the low-concentrations at the upstream end of the profile, it would be reasonable to ignore data for very low concentrations. However, the author's reason for using only the data for concentrations above, about 200 ppm (as opposed to say 50 ppm), is not clear. The divergence of the upstream data points from an approximate constant exponential (straight line) relation does not appear to be the basis for their exclusion, because the two profiles not shown by the author do not satisfy very well the condition of constant velocity and eddy coefficient as used in his application of Eq. 16. Since any interpretation would be quite arbitrary, it is requested that the author cite the values of eddy coefficient used in his calculations.

TABLE 2.—STATION RELATIONS, DELAWARE RIVER

Location (1)	Fig. 4, O'Connor, approximate miles (2)	State of Delaware Water pollution Commission, miles (3)	C. of E. (and Kent), approximate, 1000's of feet (4)
Smyrna River	24.8	0.00[a]	+305
Appoquinimink Creek	19.5	5.30[a]	+277
Reedy Island	15.5	9.28[a]	+256
Pea Patch Island	9.4	15.38[a]	+224
New Castle	3.6	21.18[a]	+193
Cherry Island	. . .	26.00[a]	+168
State Line	. . .	33.69[a]	+127
Allegheny avenue (Phila.)	. . .	57.52	0 + 0
Trenton (head of tide)	. . .	87.51	-160

[a] Stations at which samples were taken on the three 1956 Surveys partially used by O'Connor.

The phrase "probable error from statistical analysis of cross sectional data" appears as a footnote in Fig. 3. It would be highly appropriate for the author to state the statistical significance of the ranges plotted in the several figures. Because the various constants were somewhat arbitrarily assigned, a question immediately arises as to what the effect would have been on the computed oxygen profiles had other values been selected because the good agreement may have been achieved by serendipity. The statement in the Synopsis, "The assumption of constant coefficients in the equation is confirmed by the field data . . . ," presumably was not intended to include the eddy coefficient for the Delaware River.

The State of Delaware Water Pollution Commission, in cooperation with Incodel, has made twenty-three "same slack" runs in the June through October period of the intervening three years, 1957-1959. Of these, eighteen were at LWS and five at HWS, with two of the twenty-three on the same day. All of these runs extended from Fieldsboro at about Station -127 to Reedy Island at about Station +256, covering the section of the estuary of major oxygen concentration interest by means of fifteen sampling locations. The minimum D. O. for the twenty-three runs was observed at or upstream from the State Line, about Station +127. However, a chloride concentration equal to or greater than 50 ppm extended only as far as about Stations +100 to +130 for the HWS observations and from about 0+0 to +190 for the LWS. The minimum D. O. was thus well above the principal salinity front for many of the runs. In order that the author's procedures could be utilized to compute complete oxygen profiles, it would be necessary to integrate an instantaneous release or to employ a continuous release of tracer above the reach to be sampled. This has been implied in the quoted paragraph in the beginning of this discussion.

TABLE 3.—CHLORIDE DATA FOR 1956 DELAWARE RIVER SURVEYS,[a] SURFACE, SHIP CHANNEL

State of Delaware W.P.C., Miles	Station From Philadelphia Approximate	CHLORIDES, in ppm		
		Aug. 23, 1956	Oct. 9, 1956	Nov. 13, 1956
0	+305	4,857	5,250	6,900
5.3	+277	3,733	3,400	4,500
9.28	+256	2,340	2,470	2,850
15.38	+224	1,270	1,420	1,300
21.18	+193	603	810	320
26.00	+168	177	273	75
33.69	+127	60	58	18
Current		LWS	LWS	HWS
"Trenton Flow," in cfs		5,130	5,540	6,500
Q, O'Connor, Table 1, in cfs		(5,300)	(7,450)	(11,600)
Trenton Flow, USGS, Date of Survey, in cfs		(4,630)	(5,140)	(6,100)

[a] "Investigation of Sanitary Water Quality in the Lower Delaware River," by A. J. Kaplovsky, Tech. Report II, Water Pollution Comm., State of Del., December, 1956.

Returning to the question of constancy of eddy coefficient, in Fig. 8 are graphed surface salinity profiles obtained from the Delaware River model at

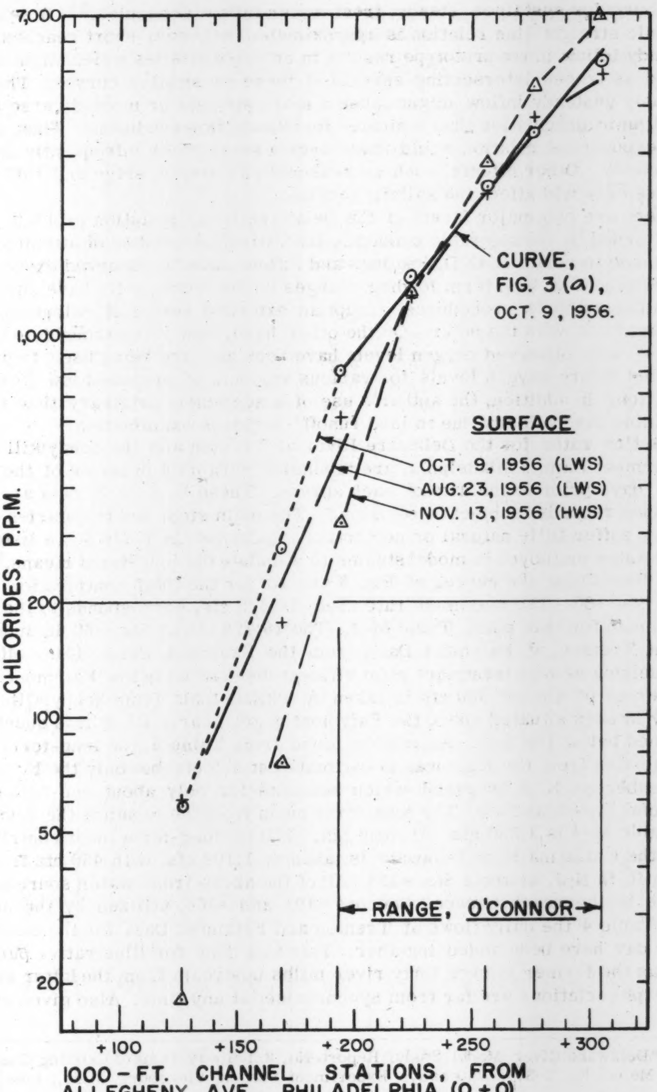

FIG. 7.—SALINITY PROFILES FOR DELAWARE RIVER SURVEYS OF FIG. 4.

Vicksburg for sustained, steady, fresh water inflow schedules.[31] A semilogarithmic straight line relation is approximated only over short reaches. The unsteady inflow in the prototype results in chloride profiles which might be regarded as traces intersecting several of these or similar curves. The continuously unsteady inflow might cause a more straight or more diverse semilogarithmic distribution than indicated for steady flow conditions. Thus, a constant exponential relation would obtain over a short reach infrequently and unpredictably. Other effects, such as seasonal and storm, stage and tide range changes,[32] would affect the salinity profile.

There are two major facets of the Delaware River pollution problem which are attended by considerable economic liabilities. A number of attempts have been made to relate B. O. D. loadings and inflow rates to observed oxygen levels. Seasonal or long term loading changes do not necessarily have any bearing on the author's procedures, except an extended series of calculations be compared one with the other. On the other hand, flow rates thought to be associated with observed oxygen levels have been and are being used to project expected future oxygen levels for various amounts of proposed low flow augmentation. In addition, the author's use of a seemingly arbitrary flow rate to determine the "velocity due to land runoff" requires examination.

The flow rates for the Delaware River at Trenton and the Schuylkill River at Fairmont Dam, Philadelphia, are tabulated in Table 4 in terms of the number of days prior to the date of each survey. These U. S. G. S. data are "Unpublished records, subject to revision." The main stem and tributaries to the estuary suffer little natural or controlled regulation. In Table 5 are listed the inflow rates employed in model studies to simulate the long-term means.[9] The rates identifying the curves of Fig. 8 are all for the total contribution above about Sta. +60. The maximum rate used, 16,475 cfs, corresponds to the long-term mean for that point, Table M-4. The 16,475 cfs at Sta. +60 includes the flow at Trenton, at Fairmont Dam, from the Rancocas Creek (Sta. -40) and Philadelphia sewage treatment plant effluent discharged below Fairmont Dam. An average of almost 300 cfs is taken by Philadelphia from Schuylkill water supply intakes situated above the Fairmont gage; nearly all of this quantity is returned below the dam. As may be found from Table 5, the long-term mean contribution from the Rancocas is estimated at 875 cfs, but only the N. Branch at Pemberton, N. J. is gaged, which accounts for only about one-fifth of the true total Rancocas flow. The long-term mean from the mouth of the Schuylkill for Table M-4 is 3,250 cfs. At about Sta. +170 the long-term mean contribution from the Christina R. in Delaware is taken at 1,100 cfs, with 450 cfs from the Salem R. in N. J. at about Sta. +235. All of the above fresh water sources contribute to the reach between Stations +193 and +305, utilized by the author.

In Table 4 the daily flows at Trenton and Fairmont Dam for the same calendar day have been added together. This was done for illustrative purposes only, as the former is over forty river miles upstream from the latter and hydrograph variations are far from synchronised at any time. Also given are the

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[31] "Delaware River Model Study, Report No. 2, Salinity Tests of Existing Channel," Tech. Memo. No. 2-337, Waterways Experiment Station, Vicksburg, Miss., June, 1954, Plates 13-18.

[32] "Salinity of the Delaware Estuary," by Bernard Cohen, U. S. Geological Survey Open File Report, Philadelphia, Penna., July, 1957.

TABLE 4.—COMBINED FLOWS FOR DELAWARE RIVER AT TRENTON AND SCHUYLKILL RIVER AT FAIRMONT DAM.[a]

Number of days prior to date of Survey	Date of Sampling Survey								
	Aug. 23, 1956			Oct. 9, 1956			Nov. 13, 1956		
	Flow at trenton gage, Delaware River, in cfs	Flow at Fairmont Dam, Philadelphia, Schuylkill River in cfs	Total flow, not including Rancocas Creek, in cfs	Flow at trenton gage, Delaware River, in cfs	Flow at Fairmont dam, Philadelphia, Schuylkill River in cfs	Total flow, not including Rancocas Creek, in cfs	Flow at trenton gage, Delaware River, in cfs	Flow at Fairmont Dam, Philadelphia, Schuylkill River in cfs	Total flow, not including Rancocas Creek, in cfs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0	4,630	1,290	5,920	5,140	929	6,069	6,100	1,910	8,010
1	4,740	1,910	6,650	5,420	1,200	6,620	6,700	2,050	8,750
2	3,900	1,270	5,170	4,700	1,120	5,820	7,350	2,290	9,640
3	3,500	410	3,910	4,630	892	5,522	7,600	2,590	10,190
4	3,530	430	3,960	4,420	892	5,312	7,950	2,830	10,780
5	3,620	470	4,090	4,420	855	5,275	8,650	3,240	11,890
6	3,680	510	4,190	4,630	818	5,448	9,500	3,820	13,320
7	3,860	670	4,530	4,980	892	5,872	10,700	4,500	15,200
8	4,000	670	4,670	5,300	966	6,266	13,000	5,800	18,800
9	3,720	606	4,326	5,860	1,080	6,940	14,100	7,700	21,800
10	3,760	606	4,366	6,550	1,120	7,670	13,300	13,000	26,300
11	3,860	744	4,604	7,120	1,290	8,410	12,200	7,700	19,900
12	4,100	744	4,844	8,950	1,000	9,950	7,120	2,100	9,220
13	4,280	892	5,172	7,840	1,200	9,040	5,740	1,500	7,240
14	4,520	1,080	5,600	5,500	1,200	6,700	5,700	1,770	7,470
15	4,820	1,388	6,208	5,540	1,050	6,590	5,820	2,150	7,970
16	4,600	1,388	5,988	6,180	920	7,100	6,140	2,390	8,530
17	4,520	1,290	5,810	5,100	980	6,080	6,700	2,680	9,380
18	4,490	929	5,419	5,700	910	6,610	7,800	3,290	11,090
19	4,630	781	5,411	6,550	980	7,530	9,200	4,800	14,000
20	4,780	818	5,598	5,540	1,040	6,580	10,100	6,070	16,170
21	4,980	855	5,835	4,600	1,200	5,800	4,920	1,770	6,690
22	5,100	966	6,066	4,490	1,450	5,940	3,900	574	4,474
23	5,340	1,120	6,460	4,100	1,100	5,200	3,930	542	4,472
24	5,980	1,370	7,350	4,000	1,030	5,030	4,000	542	4,542
25	6,920	1,630	8,550	4,100	1,100	5,200	4,040	638	4,678
26	6,960	1,580	8,540	4,280	1,200	5,480	4,140	638	4,778
27	6,500	1,290	7,790	4,520	1,350	5,870	4,100	638	4,738
28	7,120	1,500	8,620	4,740	1,500	6,240	4,000	707	4,707
29	7,950	1,860	9,810	5,660	1,860	7,520	3,960	670	4,630
Average 10 days	3,918	824	4,742	4,950	964	5,914	9,165	3,673	12,838
Average 20 days	4,138	904	5,042	5,726	1,015	6,741	8,569	3,905	12,474
Average 30 days	4,813	1,036	5,849	5,352	1,104	6,456	7,282	3,030	10,312

[a] Not including flow from Rancocas Creek, surveys of Fig. 4 and Table 1.

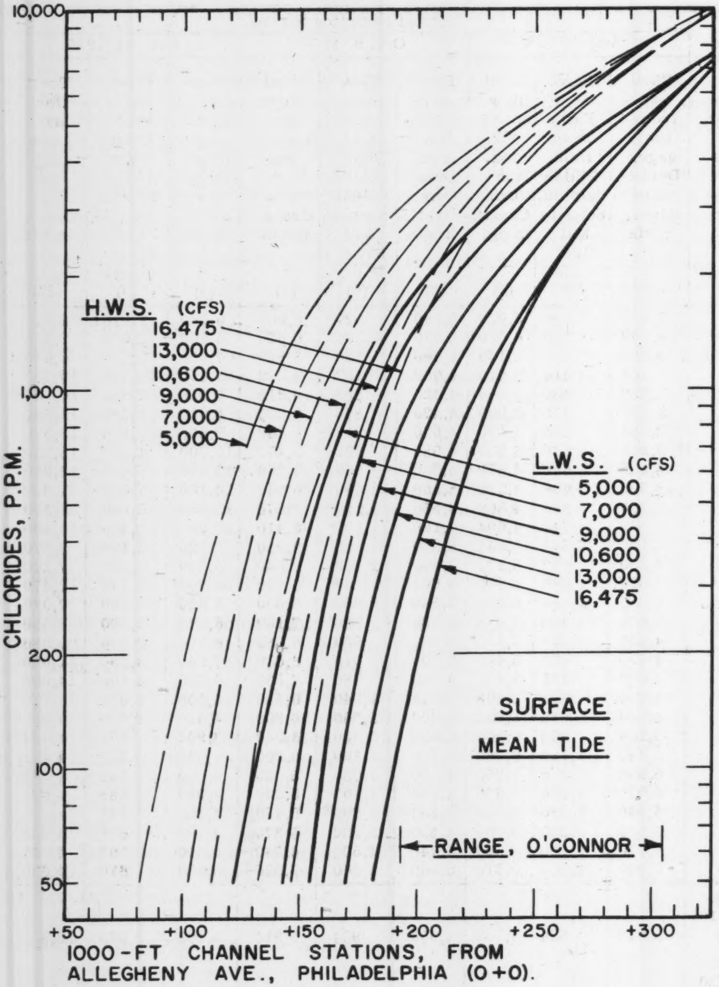

FIG. 8.—SALINITY PROFILES FOR VARIOUS RATES OF STEADY FRESH WATER INFLOW—AT AND INCLUDING THE SCHUYLKILL RIVER

corresponding 10-day, 20-day and 30-day averages for periods prior to an including the date of survey, for comparison. At the bottom of Table 3 are listed the flows given elsewhere[22] and those in Table 2. How did the author arrive at the figures used? Was the flow for the Delaware surveys indeed "low?"

The author states that: "The basic relationships defining the processes of deoxygenation and reaeration in rivers must be related to the flushing times in order to define the oxygen balance in an estuary." It is not clear whether or not flushing times were a consideration in the author's calculations. If the approximate time observed for the movement of the centroid of a dye profile in the model under steady flow conditions may be used as an index of travel time for fresh water inflow,[30] a flow at Trenton of 12,350 cfs (schedule of Table 5) would take about 21 days at LWS and 27 days at HWS to travel from Trenton to Sta. +193. With 5,000 cfs at Trenton the travel time would be about 52 days at LWS and 66 days at HWS. With these approximations in mind and considering that only thirty days are represented in Table 4, one is inclined to question seriously the premise: "A steady state condition is assumed to exist which implies that the land runoff and the waste discharge have remained constant for a period equivalent to the time of passage through the estuary."

TABLE 5.—LONG TERM MEANS, TOTAL FRESH WATER INFLOW AT VARIOUS STATIONS USED IN MODEL STUDIES, DELAWARE RIVER ESTUARY

Approximate Station (1)	Total fresh water flow, in cfs (2)
-160 (Trenton)	12,350
- 40	13,225
+ 60	16,475
+170	17,575
+235	18,025
+340	18,400
+420 (To Capes)	20,200

Range, O'Connor, Approximately Stations +193 to +305

While the reader is entitled to some explanation as to the meaning of "land runoff" and the procedures used in arriving at the figures cited, the author obviously and unquestionably is not responsible for the absence of an adequate theory or rationale for appraising inflow, particularly where extrapolation to presumed low flow augmentation conditions is an issue. However, the value of all attempts to analyze estuarine oxygen balances will be extremely limited until such time as the contributing effects from inflow can be separated. The complexity of gaging directly the flow of a tidal stream may be appreciated from the following example: In the vicinity of Sta. +127 the total long-term mean fresh water inflow is as large as only about 8% of either the total mean flood discharge or the total mean ebb discharge;[33] the critical drought flows are obviously so small as to be inseparable. A time-synchronized integration of fresh water inflow sources, using steady state transit times obtained from the model, requires the imposition of a conceptual mechanism for the natural unsteady state, along with seriously restrictive boundary conditions. Perhaps the most logical and feasible approach would be by means of the existing hydraulic model, to test the appropriateness of the several bases through which the unsteady case might be analyzed. It is the writer's firm opinion that oxygen balance analysis will not reach the degree of precision and adequacy urgently needed until the true contribution, large or small, of fresh water inflow can be appraised.

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[33] "The Prototype and Model Delaware Estuary," by C. F. Wicker, Proceedings, 6th Genl, Meeting Internatl., Assoc. of Hyd. Res., A 12-1, The Hague, The Netherlands, 1955, (Table VIII).

The published data[22] for the Delaware River surveys is given in Table C. Only the October 9, 1956 B. O. D. data were presented by the author, Fig. 3(b). The B. O. D. data for the other two surveys are more divergent.

The report of the Public Health Service seminar of 1958[34] properly could have been added to the generous bibliography in the paper. An important reference on the regression analysis approach has recently been brought to the writer's attention.[35]

TABLE 6.—DATA FOR 1956 DELAWARE RIVER SURVEYS,[a] SURFACE, SHIP CHANNEL[b]

Stations From Philadelphia, Approximate, 1000's of feet	Aug. 23, 1956			Oct. 9, 1956			Nov. 13, 1956		
	Temperature, in °C	D. O. Sat., in %	(5-Day) B.O.D., in ppm	Temperature, in °C	D. O. Sat., in %	(5-Day) B.O.D., in ppm	Temperature, in °C	D. O. Sat., in %	(5-Day) B.O.D., in ppm
+305	25.0	85.0	3.07	17	80.8	2.4	12.0	88.0	4.2
+277	25.0	91.7	2.47	17	77.7	1.67	12.0	83.0	7.4
+256	25.0	83.3	1.60	17	72.2	2.4	12.0	81.0	4.6
+224	25.5	68.4	1.20	17	56.5	4.2	12.0	68.4	5.2
+193	25.5	48.3	2.50	17.5	41.8	7.6	12.0	48.3	10.8
+168	26.0	38.7	0.70	18	33.8	9.6	13.0	42.0	15.6
+127	26.0	43.3	6.13	18	29.4	13.8	13.0	38.5	3.0

[a] "Investigation of Sanitary Water Quality in the Lower Delaware River," by A. J. Kaplovsky, Tech. Report II, Water Pollution Comm., State of Del., December, 1956.

[b] Chlorides listed in Table 3.

C. H. J. Hull,[36] M. ASCE has presented a simplified technique for determining the self-purification of fresh water streams in which the need for determining coefficients of deoxygenation and reaeration or time of flow is eliminated. It is possible that this technique may be adaptable to estuarine analyses using data for the same tidal phase. Temperature, D.O. and B.O.D. are the only measurements required.

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[34] "Oxygen Relationships in Streams," a Seminar, The Taft Sanitary Engrg. Center, Technical Report W58-2, Cincinnati, Ohio, March, 1958.

[35] "A Three Cycle Analysis of Water Quality Variables in Tidal Estuaries," by R. V. Thomann, A. N. Diachishin, P. DeFalco, Jr., and L. M. Klashman, AAAS International Oceanographic Congress, Preprints, Washington, 31 August-12 September, 1959, p. 709.

[36] "Report No. 111 of the Low-Flow Augmentation Project," by C. H. J. Hull, The Johns Hopkins University, Baltimore, Md., April, 1960.

C. H. J. HULL.[37]—Mr. O'Connor has presented a theoretical analysis purporting to define the processes which determine the distribution of dissolved oxygen in a non-stratified estuary. The author merits praise and encouragement in his efforts to unravel the extremely complex relationships involving the use, replenishment, and movement of oxygen in tidal waters. The reasonably good correlations between the computed and observed DO profiles for the Delaware and James Rivers tend to support the use of the formulations presented for purposes of determining treatment requirements of wastes to be discharged into such waters. The correlations also tend to justify using Mr. O'Connor's relationships for determining the minimum fresh-water flow requirements into estuaries such as the Delaware in which the land runoff is regulated by diversions and storage projects.

On the other hand, studies and experience by this writer lead him to question the validity of certain assumptions implicit in the author's analysis if his method is to be applied generally to any non-stratified estuary for any season or time of day. These assumptions are: (a) that the temperature dependency of the deoxygenation coefficient is truly as shown by the relationship (Eq. 21) found empirically by H. W. Streeter and E. B. Phelps (53)[38] in their Ohio River studies; and (b) that the only means of replenishing the oxygen used in the stabilization of organic matter is by the transfer of atmospheric oxygen across the air-water interface.

In view of the long standing and wide acceptance of these two assumptions among sanitary engineers, it requires some audacity to question them. As Benjamin Franklin (9) put it two hundred years ago,

> "'Tis a kind of audacity to call such general opinions in question, and may call one to censure. But we must hazard something in what we think the cause of truth: and if we propose our objections modestly we shall tho' mistaken, deserve a censure less severe, than when we are both mistaken and insolent . . ."

(Coincidentally, this quotation was from Franklin's discussion of the movement, or lack of movement, of fresh water through an estuary to the sea. As a matter of historical interest, the entire discussion is recommended to those interested in estuarine hydrology).

Actually, considerable evidence is readily available to show that the first assumption is not strictly correct. As for the second, the published scientific literature is replete with evidence to the contrary, so much of it that it becomes difficult to overlook or ignore. It should help to examine briefly the evidence on both the temperature dependency of the deoxygenation coefficient and the sources of oxygen found dissolved in natural surface waters.

Temperature.—Recently, in connection with an investigation of low-flow augmentation for pollution abatement, the writer found it impossible to obtain agreement between DO profiles observed in a stream in winter and those predicted by transposing the deoxygenation coefficient determined in summer to winter temperatures by means of the temperature-correction formula cited by the author (Eq. 21). This led to a review of the technical literature on the effects of temperature on BOD rates, as reported by Hull and Carbaugh (21).

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[38] Numerals in parentheses refer to corresponding items in the list of References located at the end of this discussion.

Briefly, the review indicated that the formulation, Eq. 21, derived by Messrs. Streeter and Phelps (53) and purportedly confirmed by E. J. Theriault (54) is not adequately supported by experimental data. The thermal-correction coefficient, θ, in the relationship

$$K_{T'} = K_T\ \theta^{(T' - T)} \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

is apparently not really a constant with the value of 1.047, as hypothesized by those early investigators. Examination of their data along with those of E. W. Moore (29) and H. B. Gotaas (13) strongly suggests that θ itself is a continuously varying function of temperature. The coefficient, θ, appears to vary non-linearly, decreasing from about 1.15 for temperatures near the freezing point to about 0.97 for temperatures near 35°C. Fortunately, the curve of θ versus temperature passes through the value of 1.047 in the range of summer temperatures normal for most surface waters. This justifies use of Eq. 22 with a θ value of 1.047 for approximating temperature corrections over the summer range from about 18°C to 27°C, although the refinement indicated by three decimal places is hardly supported by available data. However, for regulated streams where winter temperatures are involved, or in streams heated to 30°C or higher by thermal pollution, the use of the Streeter-Phelps temperature-correction coefficient of 1.047 does not appear to be justified. Therefore, in these situations, Eq. 21 must be used with caution. It would be possible, of course, to determine the deoxygenation coefficient empirically at the desired temperature by carrying out the necessary field work in the time of year concerned, thus eliminating the need to transpose the coefficient by the use of Eq. 21.

The writer does not intend to imply that until now the value of 1.047 for θ has been universally accepted without question. Earlier writers, in addition to Messrs. Moore and Gotaas, already cited, have mentioned the doubtful nature of the knowledge of temperature effects on self-purification. M. LeBosquet, Jr. and E. C. Tsivoglou (27) pointed out the need for more exhaustive investigation of temperature-correction coefficients. W. M. Ingram and W. W. Towne (22), in discussing the effects of the heating of natural waters by industrial wastes, noted that the influence of such heating on the metabolic processes of biota as reflected in measurements of oxygen demand is not fully understood, although long recognized. In spite of these and other cautioning statements which have appeared, and the available evidence to the contrary, Eq. 21 and the thermal-correction coefficient of 1.047 continue to be used for engineering purposes. While the lack of a better method of determining temperature effects tends to justify and necessitate using Eq. 21 at present, the weakness of the support for this method calls for further research. In the meantime, it appears unwise to place unlimited confidence in self-purification computations based on this weak foundation.

Sources of Oxygen.—This writer has previously stressed the fallacy of assuming only the atmosphere as a source of oxygen for waste stabilization (C. H. J. Hull 18, 19). The production of oxygen in natural waters by photosynthesis of phytoplankton and benthic algae is recognized by all limnologists, oceanographers, biologists, and sanitary engineers. However, among these various fields of overlapping interests, the sanitary engineers have been uniquely reluctant to attempt quantitative evaluation of this factor for use in estimating the self-purification capacity of surface waters. Although most engineers writing on this subject have recognized that photosynthesis may be significant as a source of oxygen, they have generally mentioned it only in vague, qualitative, short statements of a sentence or two. They have attempted to justify their lack of attention to photosynthetic oxygen by referring to the difficulty of accurately determining this factor, or by pointing out that it is an undependable source, or that the oxygen produced is subsequently consumed by algal respiration and therefore of no importance.

As for difficulty, quantitative estimation of photosynthesis by the light-and-dark-bottle technique used by Gaarder and Gran (10) in Oslo Fiord, by S. M. Marshall and A. P. Orr (28) in the sea, and by H. A. Schomer and C. Juday (49) in Wisconsin Lakes is probably no more involved than the determination of the coefficient of eddy diffusivity or of the coefficient of atmospheric reaeration (if, indeed, the latter can be determined at all during daylight hours without knowledge of the photosynthetic oxygen-production rates). Newer methods have been proposed, some of which promise to be relatively simple and reasonably accurate (H. T. Odum, 33; J. H. Ryther, 47; J. Verduin, 57).

The argument that photosynthesis is not a dependable source of oxygen for natural streams appears incompatible with the increasing dependence being placed upon photosynthesis in sewage oxidation ponds. These ponds are gaining widespread favor as a means of protecting streams from excessive pollution. In this case, engineers have attempted to measure photosynthesis as a purification factor (W. J. Oswald and H. B. Gotaas, 34). If dependable in artificial ponds, there appears to be no reason to believe that photosynthesis is less dependable in natural water bodies.

Even if not dependable in the sense that it is uniformly productive day by day, the statistical significance of photosynthesis is no less than that of variable stream flows, variable waste loads, or variable atmospheric reaeration caused by variable winds and—of all things—by the variation of the dissolved-oxygen deficit attributable to the nondependability of photosynthesis.

If photosynthetic oxygen production rates are only at times significant, then special care must be taken in oxygen-balance studies to avoid the error of crediting these undependable photosynthetic reoxygenation rates to atmospheric reaeration. Unless photosynthesis is taken into account, the reaeration coefficients computed from observed dissolved-oxygen data will automatically include a photosynthetic component. Therefore, if dependability is the criterion for a significant oxygen source worthy of study, the investigator is forced to consider the "undependable" in order to evaluate the "dependable."

As for algal respiration cancelling the benefits of photosynthesis, this appears to be a widespread opinion unsupported by experimental evidence or field observations. The sharp drop in DO concentration following sunset seems generally to have been attributed largely to algal respiration. Usually unanswered are the questions: (1) What part of this DO drop can be attributed to bacterial respiration? and (2) What part of this DO drop can be attributed to atmospheric deaeration of water supersaturated with DO after exposure to daylight for about twelve hours?

Considerable evidence is available to support the contrary opinion that, in many instances, algal photosynthesis more than balances the algal respiration. D. W. Pritchard (38) has offered the following explanation:

> "There exists a frequent mis-conception that there cannot be a net production by photosynthesis over respiration since the death and decay of

> the organisms will utilize as much oxygen as was produced in the original fixing of carbon into organic compounds. Actually, a significant portion of the dead organic material sinks to the bottom and is incorporated into the sediments there before it can exert an oxygen demand on the water. The breakdown of this organic material within the sediments takes place very slowly, and by chemical processes which do not utilize free oxygen. Some of the material remains in the organic state without ever being completely oxidized."

Mr. Pritchard's argument is supported by Franz Ruttner (46) who states that in lakes,

> ". . . the quantity of organic material per unit volume of water destined for decomposition there (hypolimnion) is indeed small in comparison to the accumulations of the same material in the sediments. . . . That these materials are able to outlast even geological epochs is demonstrated to us by sapropelite, by sedimentary bituminous deposits, and not the least by petroleum."

The slow decomposition of algae is supported also by H. W. Harvey (15), who comments that

> ". . . whole algae fall to the bottom, . . . for some odd reason the algae are only very slowly decomposed by bacteria and protozoa, although both unicellular and filamentous algae are extremely rich in protein."

Actually, to be significant in oxygen-balance computations, the oxygen demand of organic matter synthesized by algae need be delayed only a matter of a few hours in some cases. For example, in computing the reoxygenation coefficient for a short stretch of a stream using DO values observed during daylight, the time of flow through the stretch would be measured in hours, or possibly as much as a day. The fixing of carbon in the stretch would be accompanied by oxygen release. Any photosynthetic oxygen remaining in the water at the downstream end of the stretch would affect the reoxygenation coefficient if computed in the usual way. That this influence is significant is shown by the differences between daylight and darkness values of coefficients computed by M. A. Churchill (6), to be discussed subsequently. The significance has also been shown many times, of course, by the persistence for several days of DO gains in bottles exposed to light as opposed to the loss of DO in bottles kept dark in the light-and-dark-bottle technique of measuring primary production.

A. L. H. Gameson (11) recently commented on the significance of photosynthetic oxygen production measurements in the Thames estuary by A. B. Wheatland (58). Mr. Gameson pointed out that the net effect of photosynthesis depends on the ultimate fate of the decaying planktonic plants:

> ". . . if they are completely oxidized before passing out of the estuary their net effect on the oxygen balance will be slight, but if they fall to the bed of the estuary and are covered up so that oxygen cannot reach them or are removed by dredging, or if they pass out of the estuary in suspension, then the net effect during the year will be a gain in oxygen."

Mr. Gameson did not reach any conclusion concerning the fate of the carbon fixed annually in the Thames. However, it appears to the writer that the annual net effect is of far less significance than the net effect during the shorter period of critical summer conditions observed in most polluted estuaries. If photosynthesis provides a net gain of oxygen during the critical period, it is an asset in the oxygen balance. As already pointed out, moreover, a net gain over a period of a few hours can significantly affect oxygenation and deoxygenation coefficients computed from DO observations. Another way in which algae-synthesized organic carbon is removed from estuaries before decaying and exerting an oxygen demand is by entering the food chain of fin fish and shellfish which are themselves removed by commercial or sports fishermen. This is an item of no small importance in many estuaries.

In reviewing the literature on the oxygen balance of natural waters, this writer has been struck by the two different underlying opinions concerning photosynthetic oxygen production. The first, held largely by the writer's colleagues in the sanitary engineering field, is that photosynthesis is relatively unimportant and can be ignored. The second, adhered to by other groups of water scientists, is that photosynthesis as a significant source of oxygen is a long established fact and must be considered in oxygen-balance studies. Indeed, these latter experts will undoubtedly consider this discussion redundant, and wonder at the need to emphasize the importance of photosynthesis at this late date.

It should be noted that there are exceptions in sanitary engineering literature in which quantitative measurements of photosynthesis have been reported. The writer (Hull, 17), cited by C. F. Garland (12), using the light-and-dark-bottle technique, measured summer photosynthesis in the estuarine waters of Baltimore Harbor, an arm of Chesapeake Bay. The plankton algae were estimated to contribute 23.6 lb of oxygen per acre per day (2.65 g per sq m per day). This was the average production determined from 12 measurements covering the three different stations in the harbor. It is pertinent to note that this amounts to about 600,000 lb per day of photosynthetic oxygen production in the harbor, more than three times the atmospheric oxygen absorbed daily, as estimated by Garland (12). However, as Mr. Garland pointed out, Baltimore Harbor would be capable of greater atmospheric reaeration at greater oxygen deficits. Of course, photosynthesis must be credited with keeping the deficit relatively small.

I. Nusbaum and H. E. Miller (31), also using light and dark bottles, determined photosynthetic oxygen production in San Diego Bay. According to them, biological activity of chlorophyl-bearing plankton accounted for 178,500 lb per day of oxygen. By comparison, they estimated that atmospheric reaeration provided only 132,700 lb per day. These investigators noted that:

> "Attempts to determine the rate of atmospheric reaeration without taking under consideration biologic activity have resulted in the publication of vast amounts of erratic data which have been difficult to apply even to the body of water upon which the determinations were made."

Churchill (6) has presented data showing rates of oxygen recovery below impoundments for day and night conditions. Reaeration coefficients for the Watauga River were computed by M. Churchill, using the formula:

$$k_2 = \frac{\log D_A - \log D_B}{\text{Time of travel A to B}} \quad \ldots\ldots\ldots\ldots (23)$$

Comparison of daylight and darkness coefficients thus computed shows the following differences:

River Mile	Reaeration Coefficient	
	Daylight	Darkness
33.85 - 32.43	6.02	3.53
32.43 - 31.45	4.80	2.40
31.45 - 30.43	4.60	2.04

The conditions other than light for these data were identical, so that the marked differences between the daylight and darkness values of the reoxygenation coefficients can be attributed only to photosynthesis. Certainly these differences are significant.

The most recent work by engineers known to the writer which shows the significance of photosynthesis is that by R. V. Thomann et al (55). They used a three-cycle analysis to describe the variation of several water-quality characteristics of the Delaware River estuary. The three cycles are all assumed to be sinusoidal. In addition to the tidal cycle, they also considered the daily and seasonal variations. The sine curve attributed to the tidal effect has a period of slightly more than 12 hr. The daily, or diurnal, curve has a period of 24 hr. It is significant to note that the diurnal sine curve for oxygen presented as an example of the variation in the Delaware estuary (location unspecified) has a greater amplitude than the curve showing the effect of tide for the same station. Inasmuch as the 24-hr sine curve for oxygen concentration is generally the result of photosynthesis during daylight hours and its absence during darkness, it can be assumed that photosynthesis is a significant factor at that location in the estuary.

Several other references to photosynthesis appear in the technical literature familiar to sanitary engineers. Notable among these are the works of W. C. Purdy (40) on the Potomac River estuary, Purdy (41) on New York Harbor, W. Rudolfs and H. Heukelekian (45), C. K. Calvert (5), Purdy (42), G. J. Schroepfer (50), and G. M. Ridenour (43, 44). As recently as July, 1960 there appeared in this journal a most interesting paper by a group of biologists (B. W. Parker et al, 35) showing the diurnal variation of DO at a station above the tidal zone on the Delaware River. The frequent supersaturated DO values recorded are convincing evidence of photosynthetic oxygen production at that location (Riegelsville, N. J.).

Since World War II, scientists of various fields have begun working more closely with sanitary engineers in the problems of waste disposal and water-pollution control. Consequently, the subject of photosynthesis is being mentioned more and more in the engineering literature. Generally, however, the sanitary engineering publications have had little to say about primary production as a means of oxygenating natural waters.

In the literature of the other professional groups interested in water quality, one does not have to look far for information on photosynthesis. These groups have long since accepted the fact that algae and other aquatic plants contribute significant quantities of oxygen to the water. Their present concern with photosynthesis, or primary production, is directed toward means of its evaluation and measurement (J. H. Ryther et al., 48).

An extensive review of the literature of the sciences of limnology, biology, and oceangraphy on photosynthesis as it influences the oxygen balance in aquatic environments has been prepared by the writer and will be presented elsewhere (Hull, in press). This review, along with the writer's observations and measurements of primary production, has convinced the writer that photosynthesis is generally a very significant source of oxygen, and must be reckoned with in oxygen-balance studies. The almost universal occurrence of the process in water, as well as the ubiquity of algae, indicates strongly that photosynthesis is the rule rather that the exception. It therefore seems imperative, in the investigation of the oxygen balance of any water body, to consider this factor, whether the water be fresh, salt, or brackish. Without evidence other than "generally accepted opinion among sanitary engineers" to show definitely that photosynthesis is unimportant, the report of any oxygen-balance study omitting this factor must be considered incomplete, inconclusive, and probably erroneous. This applies to much of the existing literature of sanitary engineering on self-purification.

Perhaps Mr. O'Connor has negative evidence on photosynthesis in the Delaware and James estuaries. If so, it should be presented. Nevertheless, even if it can be shown with scientific observations that the neglect of photosynthesis is justified in these two estuaries, it should be emphasized that this factor cannot be assumed to be generally negligible in any estuary. The ease of determining, at least qualitatively, the occurrence or non-occurrence of photosynthesis obviates any need to rely on such an assumption without evidence.

There are several effects or products of photosynthesis which, if observed, serve to indicate the probable significance of this factor in the oxygen balance. An exhaustive search for available evidence of this kind for a given estuary is probably not justified, as it would require more time and expense than actual field measurements of primary production, and would be only qualitative. The writer has made a very hurried study of the information on the Delaware estuary available from the literature and from personal contacts with experts in possession of such information. It is emphasized that such a qualitative study is no substitute for quantitative productivity determinations.

Supporting the hypothesis that photosynthesis is significant in the polluted zone of the Delaware estuary are the following points:

(1) Plankton algae and benthic algae are present in reasonable abundance in the Delaware estuary (C. S. Boyer, 4; Public Health Service, 39; Ruth Patrick, 36). The plankton forms conceivably can be accounted for by transport into the pollution zone from fresh-water streams and from Delaware Bay. However, a healthy established population of the attached benthic forms means that photosynthesis is taking place in the polluted zone, at least in the shallow areas.

(2) Supersaturated DO values have been observed in the estuary above Philadelphia (A. J. Kaplovsky, 24), in tributaries contributing flow to the estuary (Kaplovsky, 24, Parker, et al, 35) and in the estuary below Bowers Beach, Delaware (D. P. de Sylva and F. A. Kalber, Jr., 7). Table 7 lists data showing supersaturated DO values at various locations in the Delaware estuary between Reedy Island and the Burlington-Bristol Bridge. These are taken from unpublished data furnished by the Philadelphia Water Department and represent observations of several cooperating agencies which routinely study the Delaware. The occurrence of supersaturated DO concentrations of the magnitudes shown is generally taken as positive evidence of photosynthetic activity. On the other hand, the absence of supersaturation observations be-

tween Lehigh Avenue (mile 102.16) in Philadelphia and New Castle, Delaware (mile 67.68) cannot be taken as conclusive proof that photosynthesis does not occur. It appears reasonable to assume that photosynthesis probably occurs even in this polluted zone, unless algae are inhibited there by excessive turbidity or toxicity of wastes.

(3) Mr. Kaplovsky (24) has observed that since the start of dredging to deepen the navigation channel in the Delaware River above Philadelphia, turbidity concentrations have been markedly higher than formerly recorded for the portion of the estuary between Trenton and the Philadelphia Navy Yard. At the same time, DO concentrations and pH values in the stretch have been significantly lower. These three changes, taken together, suggest the possibility that the increased turbidities have decreased light penetration and thereby lowered photosynthetic rates. It is conceivable, of course, that dredging has also increased the suspended BOD load by stirring up bottom deposits of organic matter. In this connection, however, Powell Engineers (37) found little or no sludge deposits in the Delaware estuary channel above Philadelphia. Another possibility is that the increased suspended matter caused by dredging has created a favorable environment for bacteria, many types of which are known to multiply more rapidly when solid surfaces are present (C. E. ZoBell

TABLE 7.—DELAWARE RIVER ESTUARY DATA SHOWING SUPERSATURATED D.O.[a]

River Mile (above Cape Henlopen Light)		Date, Year-Day[b]	Time 24-hr clock	Depth in feet	Chlorides, in ppm	Temp. in °C	D. O.	
							in mg/l	in % saturation
60.96-4#	(Pea Patch Island)	52-321	0936	20	4425	8.6	12.3	110
			0938	40	4555	8.6	12.5	112
				01	4295	8.7	12.2	109
54.86-4	(Reedy Island)	54-222	1624	20	4150	24.7	11.6	144
			1627	46	4105	24.7	11.1	137
			1621	01	3925	25.3	12.0	150
60.96-4	(Pea Patch Island)		1704	20	3165	25.3	10.8	134
				40	3425	25.0	10.5	130
				01	3280	25.3	11.1	138
67.68-4	(New Castle, Del.)		1744	01	2560	25.1	9.0	108
				20	2500	25.1	12.0	155
110.45-2	(Ben. Fr. Bridge)	54-067	0530	01	0017	7.0	13.4	110
54.86-4	(Reedy Island)	54-213	1338	51	4700	26.4	8.2	105
				01	4220	27.5	8.9	116
102.16-4	(Lehigh Avenue)	55-153	0854	03	11	22.8	10.0	112
117.81-3	(Burl.-Br. Bridge)	55-186	1131	03	10	28.9	8.7	112
110.45-2	(Ben. Fr. Bridge)	56-041	0730	01	--	2.0	15.2	110
117.81-2	(Burl.-Br. Bridge)	56-088	0750	01	--	5.4	14.6	116
		56-096	0715	01	--	9.0	13.0	112
117.81-4	(Burl.-Br. Bridge)	56-198	1006	03	5	23.9	10.6	123
		58-217	1335	03	5	27.5	8.9	111

[a] Unpublished data from files of Philadelphia Water Department.

[b] Days of year are numbered 1 through 366.

Numbers from 1-7 following the river mile designate sampling locations; location 1 is near west shore and 7 is near east shore of estuary.

and D. Q. Anderson, 60; Messrs. Heukelekian and Heller, 16). Hence, the suspended solids would be expected to increase the bacterial growth rate (and therefore the BOD rate) even without increased concentrations of organic matter.

(4) Millions of pounds of seafood and other fish products are removed from the Delaware estuary annually. The Delaware is considered a biologically productive estuary (F. C. June and J. Reintjes, 23; C. N. Shuster, Jr., 51). Fish are high in protein content, and protein is about 50% carbon, according to C. R. Noller (30). This organic carbon was synthesized from inorganic carbon by the primary production of chlorophyl-bearing organisms, the "grasses of the sea." Some of the fish taken in the Delaware estuary are migratory and represent carbon that was originally fixed outside the estuary. Moreover, at least part of the basic food supply attracting migratory fish into the estuary was surely not synthesized in the estuary, but was imported in sea water and in terrestrial runoff, including man-made wastes. Nevertheless, if conditions are suitable to support millions of pounds of migratory and indigenous fin and shellfish, it is reasonable to assume primary production in the estuary. Primary production is accompanied by the release of oxygen, a molecule of oxygen for every atom of carbon fixed.

On the negative side, tending to discount photosynthesis as a major source of oxygen in the Delaware estuary, is the fact that the waters in the Delaware are relatively turbid compared to other estuaries where photosynthesis is known to be an important factor. Secchi-disc observations reported by de Sylva and Kalber (7) show low water transparency.

Taking all the evidence, together with the knowledge that algae and algae-released oxygen have generally been found wherever they have been sought, it is reasonable to assume that photosynthesis is a significant source of oxygen in the Delaware estuary. At any rate, it is far safer to assume that it is than to assume it is not. Whether it is a major source, a negligible source, or something in between, can be determined only by field measurements of production rates.

It should be noted that Mr. O'Connor is certainly not unaware of the effects of algae in the oxygenation of surface waters. In a paper by D. J. O'Connor and W. E. Dobbins (32), and in published discussions of that paper, the influence of algae was noted to explain lack of agreement between various values of the reaeration coefficient computed from observed DO and BOD data and values computed with a proposed "theoretical" formula for K_2. In view of this awareness, the author's present omission of photosynthetic oxygen production from the oxygen-balance theory for an estuary is all the more puzzling.

Still other sources of oxygen which have been considered in other estuaries may be important in the Delaware and James River estuaries. Messrs. Wheatland (58) and Gameson (11) have investigated the nitrogen and sulfur cycles as possible sources of oxygen in the Thames estuary. Mr. Gameson estimated that denitrification of nitrates entering the estuary at the head of tide contributed 56 tons of oxygen per day to the oxygen balance. An additional 22 tons per day was estimated for tributaries of the Thames. Gameson also considered the gain in oxygen in the Thames estuary resulting from reduction of sulfates followed by removal of the reduced sulfates by dredging and by loss of hydrogen sulfide to the atmosphere. These losses were estimated to be equivalent to a gain in oxygen of about 16 tons per day. The total gain from denitrification and sulfate reduction was, therefore, about 94 tons of oxygen per day, a significant amount when compared to the average rate of oxygen utilization in the Thames estuary of about 700 tons per day.

Why emphasize photosynthesis and other possible sources of oxygen with such a lengthy discussion? Simply this: the only reason for proposing a theory to describe the oxygen balance is to allow one to predict the oxygen concentration for conditions other than those prevailing at any given time of observation. If the theory is correct and the correction factors are available to transpose the theory from observed conditions to the predicted conditions of temperature, dilution, waste loading, and other factors, it will be possible to forecast the oxygen concentration at a given time and place. However, if the theory is incomplete or erroneous, the predictions will be meaningless. For example, the temperature-correction coefficient for atmospheric reaeration (and even this is not known definitely) cannot be expected to apply to photosynthetic reoxygenation. Nor is there any reason to assume that the rate of change of the organic matter present as influenced by primary production will be adequately accounted for by a negative component of the coefficient of BOD removal, K_r. If photosynthesis is a significant source of oxygen and organic matter in an estuary, any theory which omits this factor cannot be used to predict DO concentrations.

If the only purpose of a mathematical theory is to fit a smooth curve through observed DO values, then Mr. O'Connor's formulas can probably be fitted reasonably well to the "sag" in most polluted estuaries, provided suitable coefficients are chosen. But if the formula does not include all factors causing a significant influence on the oxygen balance, such a smoothed curve cannot provide any more insight into the problem of waste treatment or of flow regulation than would a curve smoothed through the observed points by eye.

It has been almost forty years since W. C. Purdy (41), Kenneth Allen (1), and H. W. Streeter (22) pointed out the danger of omitting consideration of photosynthesis in a report by R. H. Gould (14) on New York Harbor. Since then, considerable evidence has been reported to indicate that photosynthesis and other factors are important. If sanitary engineers wish to continue omitting these factors from oxygen-balance studies, then it is time to begin gathering negative evidence to counteract the positive evidence accumulated over the last hundred years.

The expression for the atmospheric reaeration coefficient developed by Messrs. O'Connor and Dobbins (32) in an earlier paper and used by Mr. O'Connor in the present paper (Eq. 20) is another item to be viewed with caution. The equation for K_2, if valid, would be a most useful tool in oxygen-balance studies as it would provide a value for K_2 independent of DO, BOD, and time-of-travel measurements, which are difficult to determine accurately. However, the K_2 formula appears not to have been adequately tested. The principal support for the formula seems to be the close agreement between K_2 values calculated from the O'Connor-Dobbins equation and selected values from earlier literature calculated in the customary way using the oxygen-sag equation of Streeter and Phelps (53). It is significant to note that in selecting Streeter-Phelps values for comparison, many were discarded because of deviations presumed to be caused by algae or sludge deposits. The criteria used to judge whether algae influenced the Streeter-Phelps values of K_2 are not clear in all cases. For one of the water bodies considered, San Diego Bay, algae effects were separated by light-and-dark-bottle tests, as reported by Messrs. Nusbaum and Miller (31). Concerning the Illinois River data, it was reported by W. H. Wisely and C. W. Klassen (59) that ". . . oxygen will be added to the river by . . . photosynthesis . . ." but that photosynthesis was not considered in their report. Messrs. O'Connor and Dobbins (32) stated that variation of DO concentrations at one station in the Illinois River were small, presumptive (but not conclusive) evidence that algae effects were negligible. For most of the streams used in the verification of the theory, however, no evidence was presented for or against algae influence. For one of these streams, the Ohio River, A. F. Bartsch (3) has since presented light-and-dark bottle evidence of considerable reoxygenation by photosynthesis. For another, the Illinois River, evidence of significant algae production has been given by C. A. Kofoid (26). In view of the evidence for the influence of photosynthesis in some of the waters, and the lack of evidence against algal effects in other waters considered by Messrs. O'Connor and Dobbins in the verification of the theoretical formula for K_2, this formula cannot yet be considered verified.

The K_2 formula (Eq. 20) should not be discarded, for such a device is urgently needed. However, before using it as a basis for a more complex theory of oxygen balance, the K_2 formula should be thoroughly tested under rigidly controlled conditions which positively eliminate photosynthesis, not to mention denitrification, sulfate reduction, and the potential differences between the coefficient of BOD removal and the deoxygenation coefficient. All of these factors are potential sources of error in the K_2 values computed from the oxygen-sag formula and used in the verification of the O'Connor-Dobbins relationship.

Because of the currently unsettled controversy over the solubility of oxygen in natural waters, as shown by the discrepancies among values reported by the American Public Health Association (2), G. A. Truesdale, A. L. Downing, and G. F. Lowden (56), and H. L. Elmore and T. W. Hayes (8), it would be helpful if all studies of oxygen balance would clearly specify which solubility tables were used. This would allow later evaluation and revision of the reports if and when the true solubility values are determined. It should be noted that an extensive research project is now underway at the Chesapeake Bay Institute of The Johns Hopkins University to resolve the reported differences in oxygen solubility.

The author has labeled some of the curves as Flood slack and Ebb slack. It would be preferable to conform consistently to the accepted, nonambiguous phraseology, high-water slack and low-water slack.

In order to distinguish the coefficient K_2, it is recommended that the definition of this symbol given in Appendix II be changed to read "coefficient of atmospheric reaeration." This would prevent confusion with the reoxygenating effects of photosynthesis and other factors.

In view of the findings of Messrs. Wheatland (58) and Gameson (11) on the reduction of sulfates in the Thames estuary, it would appear that a review is in order of Mr. O'Connor's use of sulfates as a "conservative" tracer for determination of the eddy-diffusion coefficient in the James River estuary.

Contrary to the tone of this discussion, the writer does not feel that the formulation of a theory adequate for predicting the oxygen concentrations in time and space for a natural water body is a hopeless task. However, such a theory will include more than deoxygenation by carbonaceous BOD and reoxygenation by exchange across the air-water interface. The factors which have been considered in past studies are:

Oxygen Losses.—

(1) Total respiration, including carbonaceous BOD, nitrification, benthal oxygen demand;

(2) "Advective" losses, including oxygen in water diversions and water moving seaward;

(3) Chemical oxygen demand;

(4) Atmospheric deaeration (of supersaturated surface water);

(5) Dilution by oxygen-free water; and

(6) Ebullition of gases which purge oxygen from the water.

Oxygen Gains.—

(1) Atmospheric aeration;

(2) Photosynthetic oxygenation (plankton and benthos);

(3) Advective gains (fresh water inflow, waste discharges, sea water intrusion);

(4) Denitrification; and

(5) Sulfate reduction.

A single formula relating all of these factors, together with the necessary correction coefficients for temperature, sunlight intensity, turbidity, stream flow, and other controlling variables would be extremely complex and probably not amenable to computation except by high-speed automatic computers. Fortunately, computers are now available. When engineers begin to accept the importance of factors beyond the two traditional ones, first-stage BOD and atmospheric reaeration, progress will be made toward a theory describing the oxygen balance.

Acknowledgment.—Much of this discussion is based on material in the files of the Low-Flow Augmentation Project, Department of Sanitary Engineering and Water Resources, The Johns Hopkins University. The Low-Flow Project is currently supported by Research Grant RG-5312 (C3) from the National Institutes of Health, Public Health Service.

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DONALD J. O'CONNOR,[39] M. ASCE.—Mr. Thoman is correct in stating that non-stratification has been assumed in both the lateral and vertical directions. These conditions are representative of both the James and Delaware Rivers over the section analyzed. The probable error, as determined from a statistical analysis of the cross-sectional data, does not necessarily represent any degree of stratification. Three of the ten sets of cross-sectional data of the Delaware River do indicate some lateral stratification, in the order of 0.5 ppm dissolved oxygen. The remaining seven sets indicate the variation is random and may not be assigned to a definite direction or elevation. It was therefore assumed lateral stratification was not significant. Mr. Thoman pointed out the sensitivity of Eq. 8 to the value of fresh water discharge and diffusivity. Further mathematical analysis and numerical computations have indicated the conditions under which the diffusivity is significant, the most important of which is the value of term d^2c/dx^2. At the point of inflection of the dissolved oxygen concentration curve, this term has a value of zero; in the vicinity of the inflection point, it has a low numerical value. It is for this reason that the values selected by Mr. Thomas gave similar profiles. The August profile of the Delaware and particularly the James River profiles are characterized by steeper second derivative curves and are more influenced by the value of diffusion coefficient.

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In reference to the chloride profiles, Mr. McPherson questioned the reason "for using only the data for concentrations above, about 200 (as opposed to say 50 ppm). . . ." The chloride data used were those from Station 193 to Station 305 (Corps of Engineers) or from Station 21.18 to Station 0 (State of Delaware). This stretch was selected on the basis that there were no significant sources of pollution or dilution within this distance. Thus, there was nothing arbitrary about the use of chloride data (that is, 50 ppm versus 200 ppm as a minimum); the data were used consistently for the stretch under analysis. The values of the eddy diffusivity used in the computations, as requested by Mr. McPherson, are included in both Table 1 and in each figure showing the dissolved oxygen profile. The term "probable error" that is used in its strict statistical sense, defines a range of data from the 25 percentile to the 75 percentile. The parameter was used to define the variation of the data as shown on each figure and described in the paper. The term "fresh water inflow," as used by Messers. McPherson and Thoman, is more acceptable than the term "land runoff," as used by the writer.

Mr. Hull's objection to Eq. 21 is not directed toward the form of the relationship but the value of the coefficient. It appears from both theoretical considerations and observed data that this equation is a reasonable difinition of the temperature dependency of the deoxygenation process. That the coefficient may not be 1.047 is generally acknowledged, but as Mr. Hull points out, in the approximate range from 15° to 30°, this value appears to be the best available. The writer[40] has also indicated from laboratory studies on aerated lagoons that the coefficient is 1.035 and the vast majority of reported values fall within the range of 1.030 to 1.060. The important point in this paper is that a consistent temperature dependency was incorporated into the development and use of this function permitted correlation of the available data of both BOD and dissolved oxygen profiles. In the majority of the studies with which the writer is either familar or associated, this value best fits the observed data. This criterion is far from, as Mr. Hull states, "placing unlimited confidence in self-purification computations based on this weak fundation." The writer does agree with Mr. Hull that further research is required in this respect.

The major points to which the discussions were addressed were as follows:

1. The assignment of the fresh water inflow rate.
2. The determination of the diffusivity.
3. The effect of photosynthesis.

In specific answer to the question raised by both Messers. McPherson and Thoman concerning the rate of fresh water inflow:

As stated in the section "Calculation of the Velocity Term," the rate of fresh water inflow was taken as the average value of a period of 2 weeks preceding the survey date. The discharge at Station 193 was computed by multiplying the average 2 week flow at Trenton by the ratio of the drainage area at Station 193 to that at Trenton. This ratio is 1.30, as compared to an approximate range of 1.2 to 1.4, as determined from the 10, 20 and 30 day average fresh water inflow values, given by McPherson in Table 4. The Trenton flows were obtained from the provisional records of the U. S. Geological Survey. The flow of the James River at City Point was obtained by adding the flows of the James at

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[40] "Treatment of Organic Wastes in Aerated Lagoons," by D. J. O'Connor and W. W. Eckenfelder, Journal, Water Pollution Control Federation, Vol. 32, No. 4, April, 1960.

Richmond, the Appomattox at Petersburg, and Kanawha Canal near Richmond. The basis for determination of the fresh water inflow values was described in the paper, but it is reviewed herein in more detail in view of the questions raised by the discussion. The most significant point raised by both McPherson and Thoman concerns the time of travel from Trenton to the area under consideration, that ranges from approximately 10 days to 60 days. This factor was taken qualitatively into account by using the 2-week average, that was an arbitrary assignment. It appears a monthly average would be a more appropriate value for the August and October survey periods. It is significant, however, that the differences between the 10, 20, and 30 day averages of any one set are within about 20% as determined from Table 4, and that the differences among the sets are greater than this. The writer agrees that this is an extremely important factor and should not be disregarded in either tidal or non-tidal rivers. Mr. McPherson suggested the use of the existing hydraulic model to analyze both the steady and non-steady state of fresh-water inflow. The writer fully agrees that this approach may provide the data necessary to evaluate the contribution of fresh water inflow.

Mr. McPherson questioned the constancy of the diffusion coefficient and in reference to this point presented the salinity profiles obtained from the Delaware River model at Vicksburg. Over the total reach shown in Fig. 8, there is no doubt that the diffusion coefficient is not constant. However, over the stretch under consideration in this paper, that is from Stations 193 to 305 the data indicates a reasonable constancy of this coefficient, as indicated in Fig. 9. In this figure, the orginal data are plotted rather than the curves, as shown in McPherson's Fig. 8. The data cover the range of flows from about 5000 cfs to 13,000 cfs that were used in the analyses of the oxygen balances. The straight lines defined by these data do indicate a constant value of the coefficient over the stretch under consideration, but obviously not of the entire estuary. The stretch over which the coefficient may be assumed constant is usually of sufficient length (about 20, miles to 25 miles) to cover the drop and subsequent rise in the dissolved profile. If, however, it is desired to define a profile under a steady state for a greater distance, then the analysis may proceed in stretches over which the coefficients are reasonably constant or a variable coefficient may be incorporated in the basic differential equations. The coefficients, computed by means of Eq. 16 for the data shown in Fig. 9 are plotted against fresh water inflow on logarithmic coordinates in Fig. 10. For comparison the values used in the oxygen analyses are also shown. These empirical relations indicate a relatively consistent trend for each set. However, the differences between sets, that is probably due to differences in prototype and model, is certainly a matter for further investigation. From the data on low-water slack, it appears that the value of the diffusion coefficient levels off as the flow increases. Similar data for the James River are also plotted in Fig. 10.

Mr. McPherson made reference to Mr. Hull's simplified technique[36] for determining the self purification capacity. Similar techniques have been reported, one of which is described in a paper by the writer.[34] However, this technique does not obviate the need for determining the flow rate in either estuary or river. The location of the minimum dissolved oxygen is a function of both temperature and flow. It is the writer's firm opinion that elimination of time of flow is a desired and possible achievement, but oversimplification to the point of eliminating flow is both impractical and unrealistic. The fact is emphasized that flow provides not only dilution but is the primary factor in the location of the point of the minimum dissolved oxygen. Considering the extensive arguments presented to indicate the importance of flow, with which the writer agrees in principle, it appeared inconsistent in Mr. McPherson's discussion to suggest a technique that eliminated flow.

Mr. Hull's discussion is primarily concerned with the effect of photosynthesis. Most sanitary engineers, the writer included, are not unaware of the potential contribution of oxygen by photosynthetic activity. The impressive list of references as presented by Mr. Hull, attest to the awareness of this factor. Approximately one-half of these references were by engineers or presented in engineering journals, a significant portion of which dealt with algae effects.

If the inference may be drawn from Mr. Hull's discussion that photosynthesis is always a significant factor, then the writer disagrees. If, on the other hand, Mr. Hull is emphasizing the need to consider the possible effects of algae on the dissolved oxygen profiles, then the writer fully agrees. In view of the discussion, it is perhaps appropriate to comment on some of the negative aspects of this problem, tending to discount photosynthesis as a source of oxygen.

The basic requirements for algal growth are sunlight and nutrients. In regard to the latter, F. Ruttner[41] comments

> "investigations in plant physiology in recent years have shown that besides the ten known elements essential for the growth of plants - C, H, O, N, P, S, Ca, Mg, K, Fe - a considerable number of others not only stimulate growth but are also absolutely necessary although in inconceivably small amounts."

Temperature also has a profound influence on algal growths. It was reported[42] from a study of the Illinois River that "below 45° the plankton content of the Illinois River falls to about 9% of that present at higher temperatures."

The effect of turbulence, induced by winds, waves, or flowing water appear to have an inimical effect on growth. Numerous references are given in limnological tests. G. C. Whipple[42] states that

> "some microscopic organisms are extremely fragile and are readily broken up by agitation. Of these notably the filamentous algae develop well only in fairly quiet water. . . . . this explains why many forms of algae are not found in rivers. In the larger lakes and reservoirs wind and wave action are often sufficient to destroy or prevent microscopic growths."

A further significant quotation is "In flowing streams such production is usually not as important a factor in maintaining the presence of oxygen as is the solution of atmospheric oxygen. Turbulence and other physical conditions do not favor heavy growths of these heliphilous organisms." P. S. Welch[43] states

> "systems of streams, particularly those which do not drain standing-water units of any sort, usually yield small plankton crops. Many river systems, especially the swifter ones, have been reported as producing

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[41] "Fundamentals of Limnology," by F. Ruttner, Univ. of Toronto Press, 1953.

[42] "Microscopy of Drinking Water," by G. C. Whipple, 4th Ed., John Wiley & Sons, New York, 1948.

[43] "Limnology," by P. S. Welch, 2nd Ed., McGraw-Hill Book Co. Inc., New York, 1952.

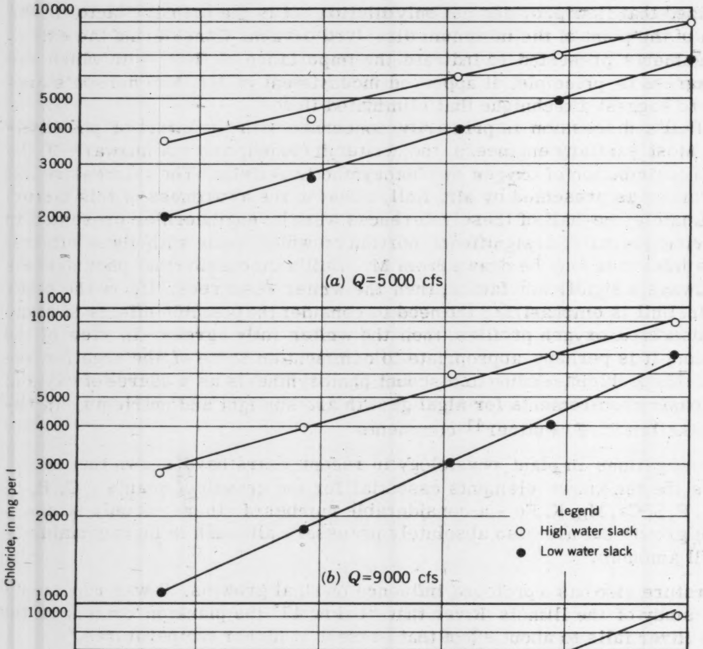

FIG. 9.—SALINITY PROFILES—DELAWARE ESTUARY

> very little plankton compared with the standing waters into which they flow" and "that water currents above a very moderate speed are distinctly inimical to plankton development."

In commenting on the communities of running water, Ruttner[41] says

> "A characteristic of clear-cut biological significance is the transport of water masses over great distances, even through contrasting climatic provinces, finally ending after a longer or shorter time with the emptying into the sea and the resulting death of most organisms carried along by the current. The duration of this transport, which is dependent on the length of the river and on the current velocity, determines whether or not there is the development of a true plankton biocoenosis - a potamoplankton. We can, indeed, capture plankton organisms with a net in many streams, particularly those containing lakes, ponds, or old river channels in their water sheds; but we do not know whether these plankton organisms are able to grow and reproduce under the altered conditions, or whether they are merely "tychoplankton" doomed to death. We can speak of a potamoplankton only when the prerequisite conditions for it have been provided, so that a special biocoenosis adapted to these particular conditions can be developed through selection of the species that are washed in. It can develop only in slowly flowing streams of great length, for example, the Volga, in which even the high water of spring requires almost two months to reach the sea. Within such periods of time, which are even longer in summer, the majority of plankton organisms find a sufficient range for the completion of their generations."

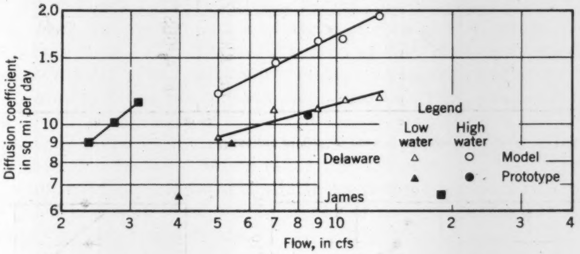

FIG. 10.—FLOW VERSUS DIFFUSION COEFFICIENT—DELAWARE AND JAMES ESTUARIES

The age of the water is reported to have a definite bearing on plankton population. It has been noted by Welch[43] that "in the upper reaches of the Sangamon River, Illinois, water approximately 9 days old contained practically no plankton and that plankton did not begin to appear abundantly until the water was at least 20 days old."

One of the conclusions the writer wishes to draw from these references is that it is highly improbable that the photosynthetic effect is present at all times

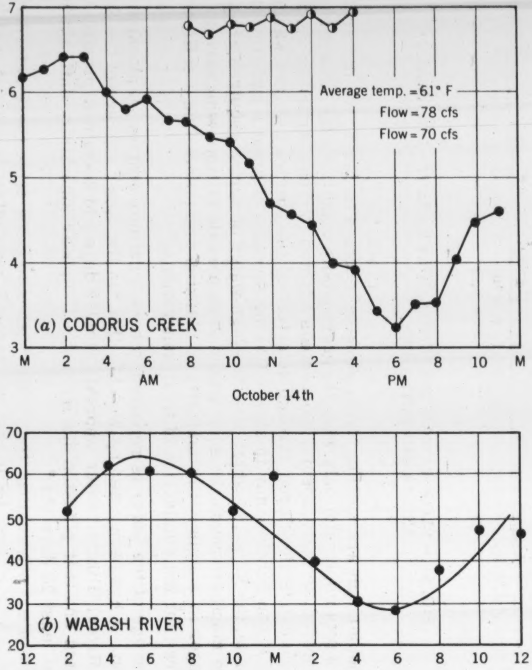

FIG. 11.—DIURNAL DISSOLVED OXYGEN VARIATION

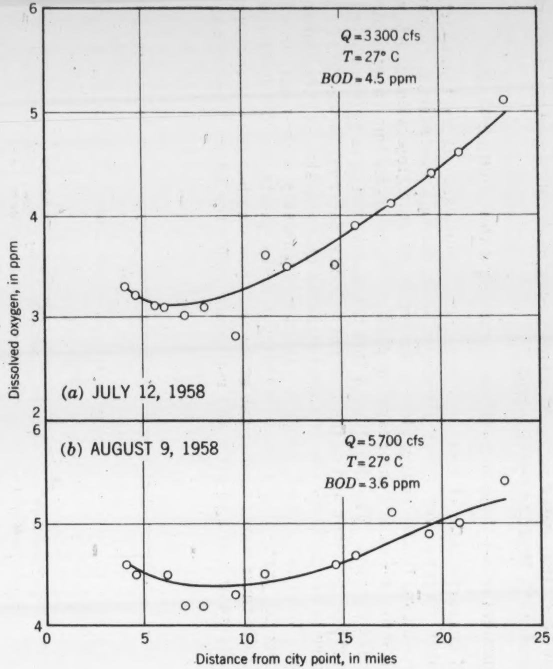

FIG. 12.—DISSOLVED OXYGEN PROFILES, 1958

and in every river. The quantitative estimation of photosynthesis by the light-and-dark bottle technique are reported by Mr. Hull inherently possesses a distinct limitation. The quiescent condition in a laboratory bottle is one that tends to promote algal activity. By contrast, the currents in natural bodies of water tend to be destructive. Thus, the increase in oxygen due to algae in the incubated and quiescent sample may not at all be indicative of actual conditions in the river.

In order to evaluate the oxygen contribution by algae, it is necessary to sample hourly for dissolved oxygen. The diurnal variation of dissolved oxygen is the best indication of this factor. Examples of such data is shown in Fig. 11 for Codorus Creek on different sampling periods. From these data, it may be concluded that the algae, if present, are not significantly contributing to the oxygen supply. By contrast, the data from the Wabash River as shown in Fig. 11 indicates the marked effect the algae have on the dissolved oxygen levels.

Mr. Hull presented data showing supersaturated dissolved oxygen concentrations in the Delaware. None of the examples presented are the same in time or place, as those used by the writer. As a matter of fact the data of 1956, reported by Kaplovsky, indicate no supersaturated values at any station during any survey, for both the corss-sectional series and the same slack series. In other words, there was no indication of algae effects and therefore, no consideration was given to it. The data referred to by Mr. Hull undoubtedly do reflect the photosynthetic effect, but they would not be appropriate to use in checking an hypothesis that does not include algae effects. This was not the purpose of the paper.

In conclusion if there is evidence to indicate the presence and effectiveness of the algae, then this factor must be taken into account. On the other hand, if there is no such evidence, it is unrealistic to assume a priori that algae are significant, particularly in view of the number of conditions that are necessary for their proper growth and activity.

The factors in the oxygen losses and gains as Mr. Hull presented them, are automatically, or may readily be, included in the present formulations either in the evaluation of the coefficients or in the actual stream sampling. The sulfate and nitrate reductions only come into play when the dissolved oxygen is zero.

The writer recently had the opportunity to analyze data of additional surveys on the James River that were conducted at both high and low water slacks. The values of the diffusion coefficient were taken from the extrapolation as shown in Fig. 10 and the other coefficients were computed in accordance with the procedures described in the paper. Two examples of the computed profiles and observed data are shown in Fig. 12.

The dissolved oxygen profiles of the Hudson River estuary and New York Harbor were analyzed in a fashion similar to that described herein. The fresh water discharge into New York Harbor was estimated in the following fashion: the Hudson River flow at Mechanicsville and the Mohawk River flow at Cohoes were added and the sum multiplied by 1.2 to allow for the tributary drainage below these stations. It had previously been estimated that for the mean summer condition it takes approximately a month for the Mechanicsville flow to reach the New York City line. Therefore a lag of 30 days was assumed between these two locations; for example, the upstream flow of July was associated with the August survey data. Because of the limited number of samples that were collected at the same slack time, average values of the monthly periods were

used in this analysis. An example of the computed profile and the observed data is shown in Fig. 13. The procedure of assigning the fresh water discharge is more in line with Messers. McPherson's and Thoman's discussions. It is also pertinent to note, in view of Mr. Hull's discussion, that a limited number of samples indicated saturated or super-saturated values in the upstream area at the city line that is relatively unpolluted. Because less than 5% of the Hudson River data indicated a possible photosynthetic effect, no attempt was made to account for this factor separately. As both Mr. McPherson and Mr. Thoman appreciated, there is a certain arbitrariness in drawing a line through a set of data. This factor is minimized when the coefficients thus determined may be correlated to a distinct independent function. In the case of the deoxygenation coefficient this function is the temperature relationship; for the reaeration coefficient it is both the temperature and velocity relationship; in the case of the

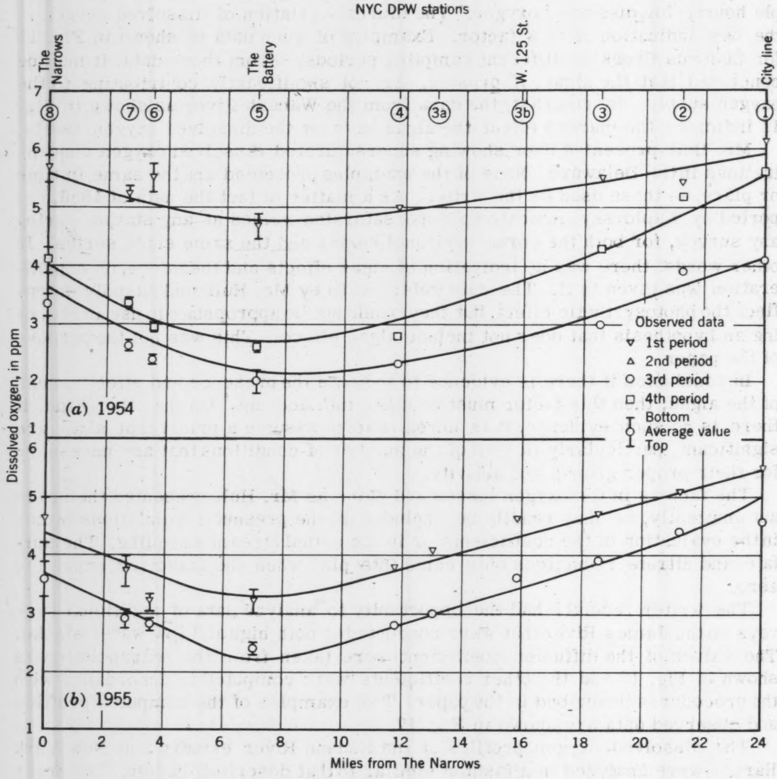

FIG. 13.—DISSOLVED OXYGEN PROFILES OF HUDSON RIVER AND UPPER HARBOR

velocity, it is the continuity equation; and some empirical relation appears to exist between flow and the diffusion coefficient.

Also, when one considers the interrelationship among the various coefficients, the arbitrariness is further minimized. Although one set of coefficients may define the dissolved oxygen profile of one set of data, these coefficients correlated to other temperatures, flow and tidal conditions must then define the dissolved oxygen profiles of other sets. There is, therefore, a consistency from one step to the next for each set of data and furthermore, a consistency in the coefficients among the various sets of data. Thus, the final curves came as a result of many trials and all coefficients used in computation of these curves were consistent within the framework of the basic relationships. Mr. Hull in his conclusion referred to the application of computers to the solution and analysis of this problem. The writer suggests that the formulation of the equation must in all cases proceed the use of the computer. The problem does not lie, as Mr. Hull states, in the refusal of engineers to accept and to understand all the factors that influence the dissolved oxygen profile; it is, rather, in our inability to construct appropriate mathematical models to define the natural phenomena that influence the profile and in our inability to express the interrelationship among these factors mathematically. When and if these ends are accomplished, the machines for both solution of the equations and analysis of data are available. But first and most important are the equations themselves. It is to this end that this paper was directed.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3234

AQUIFER TESTS IN THE SNAKE RIVER BASALT

By W. C. Walton,[1] M. ASCE, and J. W. Stewart[2]

With Discussion by Messrs. M. Maasland; and W. C. Walton

SYNOPSIS

The results of eleven aquifer tests and specific capacity data for 238 production wells indicate that the coefficient of transmissibility of the Snake River basalt ranges from 1×10^5 gpd per ft to 1.8×10^7 gpd per ft and averages about 4×10^6 gpd per ft. The coefficient of transmissibility of the entire thickness of the Snake River basalt probably greatly exceeds the values determined from test data because the wells for which data are available partially penetrate the aquifer. The coefficients of storage computed from test data are all in the water-table order of magnitude and range between 0.02 and 0.06. For the Snake River Plain, the average yield of the basalt to a 16-in. well during an 8-hr well-acceptance test, is about 2,100 gpm per ft. The aquifer ranks as one of the most productive in the United States. The average depth of well below land surface is 290 ft and the average penetration below the regional water table is 100 ft.

INTRODUCTION

The Snake River basalt is at or near the surface of and extends to considerable depths beneath most of the Snake River plain, that extends more than 300 miles across southern Idaho. Through much of this vast plain the basalt

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Note.—Published essentially as printed here, in September, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2156. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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ranges from a few hundred to possibly several thousand feet in thickness and consists of many interlocking lava flows spread out in successive sheets. The depth to the water table ranges from a few feet to more than 1,000 ft below the surface of the plain.

Each unbroken flow unit of the basalt is relatively impermeable, but porous, permeable zones along contacts between separate flows, joints, crevices, and other voids, yield and transmit large amounts of water to wells. Many individual wells penetrating 50 ft to 200 ft below the water table yield several cubic feet per second with less than a foot of drawdown.

Quantitative appraisal of the ground-water resources of the Snake River Plain requires the determination of the hydraulic properties of the basalt, the coefficients of transmissibility, and storage. One of the most important means available to the hydrologist for evaluating the hydraulic properties of an aquifer is the aquifer test, wherein the drawdown caused by pumping a well at one or more known constant rates is measured at the pumped well and at several observation wells tapping the aquifer. The formula most widely used to analyze aquifer-test data is the nonequilibrium formula of C. V. Theis[3] (1935).

The characteristics of the water-bearing openings in the basalt are not exactly as assumed in the derivation of the nonequilibrium formula. However, the results of eleven aquifer tests, that are to be presented in this report, seem to indicate that, despite the differences between assumed and actual field conditions, meaningful values for hydraulic properties can be computed by applying the nonequilibrium formula, with judgment, to the data from aquifer tests in the Snake River basalt.

NATURE OF WATER-BEARING OPENINGS

Porous, permeable zones occur along many of the contacts between adjacent basalt flows. Later flows generally do not completely fill the voids in the irregular, brecciated, and fractured surface of earlier flows. The water-bearing openings of flow-contact zones are generally interconnected, although not uniformly so in all directions, and are distributed as widely as the flows they separate.

Tension joints caused by the shrinkage of the lava as it cooled and by differential movement of the hardened crust of thick basalt flows also provide reservoirs and conduits for the storage and transmission of ground-water. The joints differ in size, and at many places form intersecting sets. Systems of joints are irregularly distributed and interconnected.

Lava poured out into water solidifies into pillowlike masses between which a great deal of open space exists. Such highly permeable pillow lavas occur at places in the Snake River basalt.

Beds of cinders are encountered in drilling in the basalt. The water-bearing characteristics of openings in the beds of cinders are similar to those of permeable sand and gravel. Cinder beds have high porosity and permeability and yield water to wells freely.

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[3] "The Relation between the Lowering of the Piezometric Surface and the Rate and Duration of Discharge of a Well Using Ground-Water Storage," by C. V. Theis, Transactions, Amer. Geophysical Union, 1935, Part 2, p. 510.

Lava tubes formed by outflow of liquid lava from beneath a hardened crust can transmit unusually large amounts of water and are often connected to other voids by joints and other fractures.

Vesicles produced by gas escaping from congealing lava and small pores between the mineral grains of the lava impart relatively high porosity to much of the Snake River basalt. The vesicles are poorly connected and the pores are so small that movement of water through them is greatly limited. However, these voids may be important to the storage capacity of the basalt.

HYDRAULIC PROPERTIES

The significant hydraulic properties of an aquifer are the coefficients of transmissibility, T, permeability, P, and storage, S. The field coefficient of permeability is defined as the rate of flow of water, in gallons per day, through a cross-sectional area of 1 sq ft of the aquifer under a hydraulic gradient of 1 ft per ft at the prevailing temperature of the ground-water. The coefficient of transmissibility is defined as the rate of flow of water, in gallons per day, through a vertical strip of the aquifer 1 ft wide and extending the full saturated thickness under a hydraulic gradient of 100% (1 ft per ft) at the prevailing temperature of the water. Thus, the coefficient of transmissibility is the field coefficient of permeability multiplied by the thickness of the aquifer, in ft. The coefficient of storage of an aquifer is defined as the volume of water it releases from or takes into storage per unit surface area of the aquifer per unit change in the component of head normal to that surface.

Permeability has little meaning in relation to an unbroken unit of the Snake River basalt. Permeable zones are concentrated along the contacts between separate flows. In addition, joints and other crevices are irregularly distributed throughout the basalt. The basalt, therefore, is heterogeneous and anisotropic (that is, its permeability varies greatly from place to place and with depth and direction of flow). The vertical permeability of the basalt is much smaller than the horizontal. Massive layers of relatively impermeable basalt and fine-grained interbeds often separate adjacent permeable zones. The hydraulic properties of any two successive zones are not likely to be the same. Wells in the Snake River basalt usually extend through two or more permeable interflow zones. During an aquifer test, adjustment of flow occurs between zones having different hydraulic properties.

Ground-water occurs under confined (artesian) conditions in some parts of the Snake River plain. For example, in the Mud Lake basin there are several flowing and nonflowing artesian wells. Over much of the plain, however, ground-water occurs under complex water-table conditions that are not yet fully understood. Under water-table conditions water is derived from storage mainly by the gravity drainage of openings in the aquifer. However, the coefficient of storage not only includes the water drained by gravity from the dewatered volume (specific yield), but also a small amount of water derived from the underlying, still saturated zone by the compaction of the aquifer, and by expansion of the water itself.

The water table beneath the Snake River plain occurs both in exceptionally porous materials (flow-contact zones, cinder beds, and so on), capable of yielding large quantities of water when allowed to drain, and in layers of massive basalt having a low specific yield. Thus, the cone of influence created by pumping a well during an aquifer test ordinarily expands into materials having different storage properties during its growth. In addition, the water-bearing openings do not drain instantaneously as assumed in the Theis formula. For these reasons, the coefficient of storage may vary with time, especially during the early parts (the first few minutes or hours of pumping, according to aquifer conditions) of an aquifer test. As time passes, water rapidly derived from storage by the gravity drainage of relatively large voids is augmented by the very slow drainage of small pores (vesicles and pores between mineral grains) and crevices. Thus, the coefficient of storage computed from the results of a relatively short test (perhaps 72 hr or less) may be much less than the true value.

Relatively impermeable layers of massive basalt and scattered layers of soil or water-laid sediments of low permeability occur both above and within the zone of saturation. These layers impede communication between the atmosphere and the water at different depths in the aquifer. As a result, ground-water in the basalt behaves as though it were under artesian pressure, in that water levels in the wells respond to changes in atmospheric pressure. Also, the upper surface of the zone of saturation occurs in layers of relatively impervious basalt as well as in porous and permeable zones. Thus, the ground water is somewhat confined, preventing the rapid decay of changes in water levels caused by fluctuations in atmospheric pressure.

AQUIFER-TEST THEORY

One of the best ways of analyzing aquifer-test data is by means of the nonequilibrium formula developed by Theis[3] (1935). The nonequilibrium formula was developed on the basis of the following assumptions: That the aquifer is infinite in areal extent and is of the same thickness throughout; that it is homogeneous and isotropic; that it is confined between impermeable beds; that the coefficient of storage is constant; that water is released from storage instantaneously with a decline in head; and that the well has an infinitesimal diameter and penetrates the entire thickness of the formation. The ideal conditions assumed in the derivation of the nonequilibrium formula do not prevail in the Snake River basalt. Therefore, application of the nonequilibrium formula to the results of tests in basalt requires judgment based on knowledge of geologic conditions in order that meaningful values for hydraulic properties can be determined. It should be recognized that the computed results of a test are only approximations of average conditions in the vicinity of the test area.

AQUIFER-TEST DATA

Aquifer tests in the Snake River basalt were made on wells at the National Reactor Testing Station (NRTS) near Arco, Idaho, in the Mud Lake basin, and in Minidoka County northwest of Rupert, Idaho. The results of two tests made at the NRTS are presented in detail to illustrate the analysis of data. The results of other tests are summarized later in this report.

An aquifer test (test 1) was made by the United States Geological Survey, Department of the Interior (USGS) in cooperation with the United States Atomic Energy Commission (AEC) on November 12-15, 1953; a group of wells

(Fig. 1) in the northern part of the NRTS area was used. The generalized graphic logs of the wells are given in Fig. 2. The effects of pumping well 6N-31E-13ac2 were measured in observation wells 6N-31E-13ac1, 6N-31E-13ca1, and 6N-31E-12ac1. Pumping was started at 12:30 p.m. on November 12, and was continued for a period of 72 hr at a constant rate of 1,220 gpm. Pumping was stopped at 12:30 p.m. on November 15 and the water level in the aquifer was allowed to recover. Fig. 3 shows the effect on water levels caused by starting and stopping the pump in well 6N-31E-13ac2.

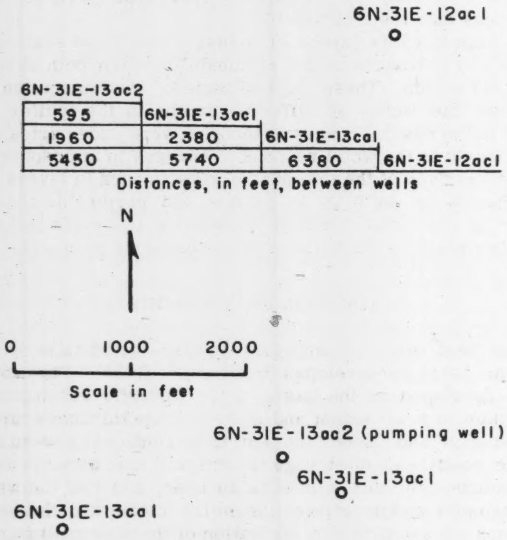
6N-31E-12ac1
6N-31E-13ac2
595
6N-31E-13ac1
1960
2380
6N-31E-13ca1
5450
5740
6380
6N-31E-12ac1
Distances, in feet, between wells
N
0
1000
2000
Scale in feet
6N-31E-13ac2 (pumping well)
6N-31E-13ac1
6N-31E-13ca1

FIG. 1.—MAP SHOWING LOCATIONS OF WELLS USED IN TEST

Drawdowns in the pumped and observation wells were determined by comparing the extrapolated graphs of water levels measured before pumping started with the graphs of water levels measured during pumping. The drawdown data were then adjusted for atmospheric-pressure changes that occurred during the test (see Fig. 3). The barometric efficiencies of the wells were determined by using water-level data collected during the latter part of the test. Adjusted drawdowns were plotted against time on logarithmic or semilogarithmic paper. The adjusted time-drawdown data based on a barometric efficiency of 88% for well 6N-31E-13ac1 are given, as an example, in Fig. 4.

The test data were analyzed by the graphical method of superposition devised by Theis and described by L. K. Wenzel[4] (1942) to solve the non-

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[4] "Methods for Determining Permeability of Water-Bearing Materials," by L. K. Wenzel, Water-Supply Paper 887, U.S.G.S., 1942.

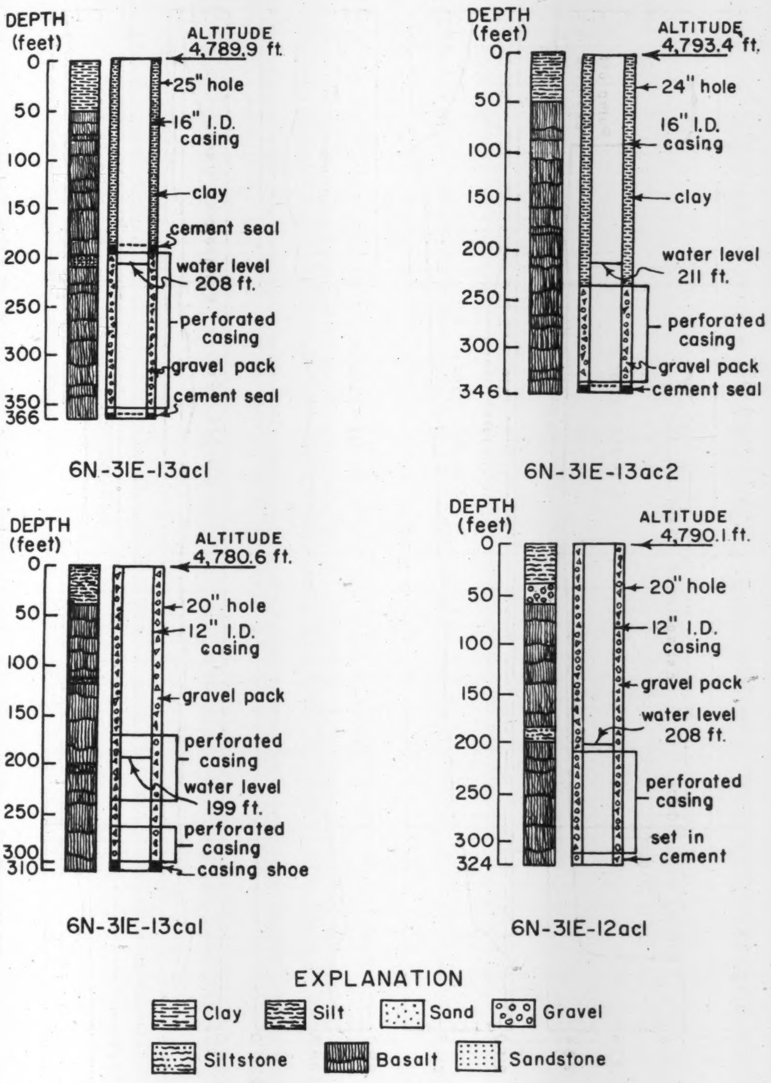

DEPTH (feet)
0
50
100
150
200
250
300
350
366
ALTITUDE 4,789.9 ft.
25" hole
16" I.D. casing
clay
cement seal
water level 208 ft.
perforated casing
gravel pack
cement seal
6N-31E-13ac1
DEPTH (feet)
0
50
100
150
200
250
300
346
ALTITUDE 4,793.4 ft.
24" hole
16" I.D. casing
clay
water level 211 ft.
perforated casing
gravel pack
cement seal
6N-31E-13ac2
DEPTH (feet)
0
50
100
150
200
250
300
310
ALTITUDE 4,780.6 ft.
20" hole
12" I.D. casing
gravel pack
perforated casing
water level 199 ft.
perforated casing
casing shoe
6N-31E-13ca1
DEPTH (feet)
0
50
100
150
200
250
300
324
ALTITUDE 4,790.1 ft.
20" hole
12" I.D. casing
gravel pack
water level 208 ft.
perforated casing
set in cement
6N-31E-12ac1
EXPLANATION
Clay
Silt
Sand
Gravel
Siltstone
Basalt
Sandstone

FIG. 2.—GENERALIZED GRAPHIC LOGS OF WELLS USED IN TEST 1

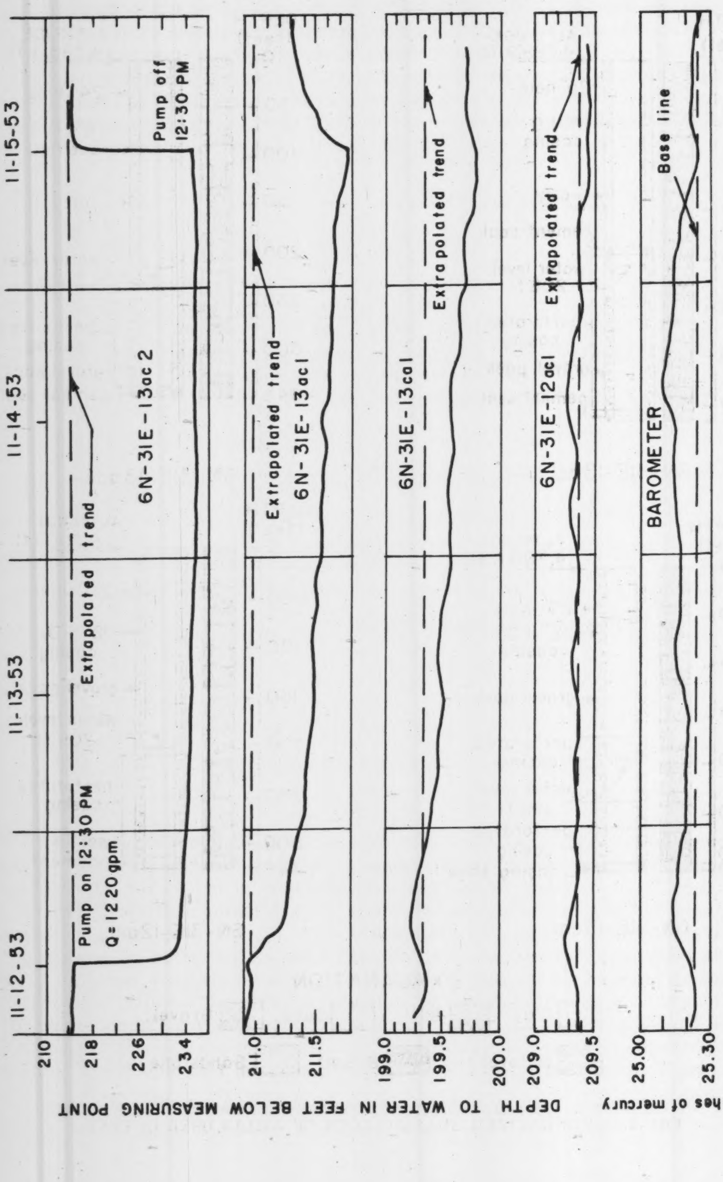


FIG. 3.—EFFECT ON WATER LEVELS CAUSED BY STARTING OR STOPPING PUMP IN WELL 6N-31E-13ac2

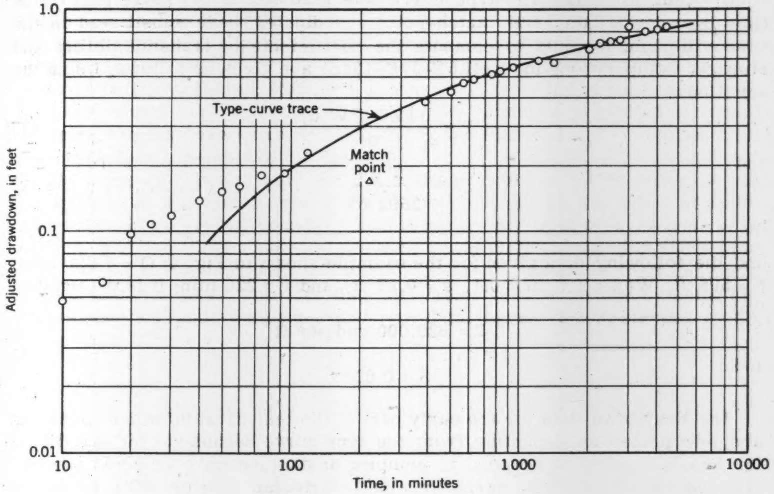


FIG. 4.—TIME-DRAWDOWN GRAPH FOR WELL 6N-31E-13ac1

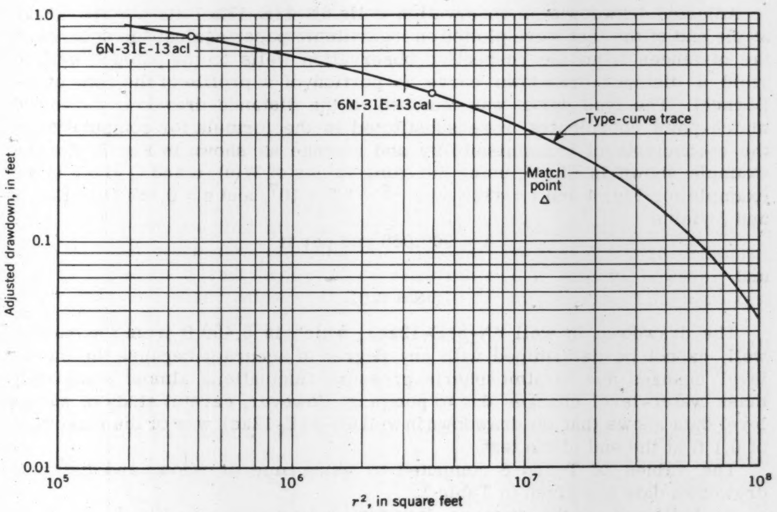


FIG. 5.—DISTANCE-DRAWDOWN GRAPH FOR TEST 1

equilibrium formula. The type curve was matched to the latter part of the time-drawdown data, and match-point coordinates were substituted in the nonequilibrium formula to compute the coefficients of transmissibility and storage. Computations for well 6N-31E-13ac1 are given as follows. Using the equations

$$T = \frac{114.6\ Q\ W(u)}{s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

and

$$S = \frac{T\ u\ t}{2693\ r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (2)$$

and the following data given for the example shown in Fig. 4: $Q = 1{,}220$ gpm, $r = 595$ ft, $W(u) = 1.0$, $u = 0.1$, $s = 0.17$ ft, and $t = 220$ min; it is determined that

$$T = 820{,}000 \text{ gpd per ft}$$

and

$$S = 0.02$$

The drawdown data for the early part of the test (first 90 min of pumping) are interpreted as departing from the type curve because of the inability of the basalt aquifer to respond to pumping instantaneously as would a homogeneous, isotropic, and perfectly elastic artesian aquifer. With prolonged pumping, adjustment of flow occurs between zones of different permeability, the effects of delay in gravity drainage become small, and the differences between the water-bearing characteristics of the basalt and ideal conditions become negligible.

Adjusted drawdowns in observation wells 6N-31E-13ac1 and 6N-31E-13ca1 at the end of the test were plotted on logarithmic paper against the squares of the distances from the respective observation wells to the pumped well, to yield a distance-drawdown curve (a portion of a profile of the cone of influence). The type curve was matched to the distance-drawdown curve and match-point coordinates were substituted in the formula for computation of the coefficients of transmissibility and storage as shown in Fig. 5. For the example shown in Fig. 5, using the same values of W(u), u and Q given in the example for Fig. 4 and $t = 4320$ min, $r^2 = 1.2 \times 10^7$, and $s = 0.155$ ft in Eqs. 1 and 2 yields

$$T = 900{,}000 \text{ gpd per ft}$$

and

$$S = 0.01$$

The drawdown in well 6N-31E-12ac1, which is 5,450 ft from the pumped well, cannot be determined with any degree of accuracy because the water-level changes due to atmospheric-pressure fluctuations almost completely mask water-level changes due to pumping. However, careful study of water-level data shows that the drawdown in well 6N-31E-12ac1 was of the magnitude of 0.1 ft at the end of the test.

The values of T and S computed by using time-drawdown and distance-drawdown data are given in Table 1.

As indicated by the distance-drawdown curve shown in Fig. 5, the 72-hr test sampled an area of basalt having a diameter of roughly 8 miles. The coefficients computed from the results of the test represent the average hydraulic properties of the basalt within that large cone of influence. Considering the complexity of ground-water conditions in the basalt, the agreement between the values of T and S computed from the results of the test is as close as can be expected. However, as is obvious from the preceding analysis, the performance of a well at any particular site cannot be predicted accurately by using the computed values of T and S, because of local irregularities in the hydraulic properties of the basalt.

The time-drawdown data did not indicate changes in the water-bearing properties of the basalt great enough to approximate the effect of an impermeable boundary. A boundary of this nature would distort the cone of influence, and the distance-drawdown data would yield values of T and S which were not in agreement with the values of T and S computed from unaffected time-drawdown data. The fact that the values of T, computed by using both time-drawdown and distance-drawdown data, agree lends support to this interpretation.

TABLE 1.—COEFFICIENTS OF TRANSMISSIBILITY AND STORAGE FOR TEST 1

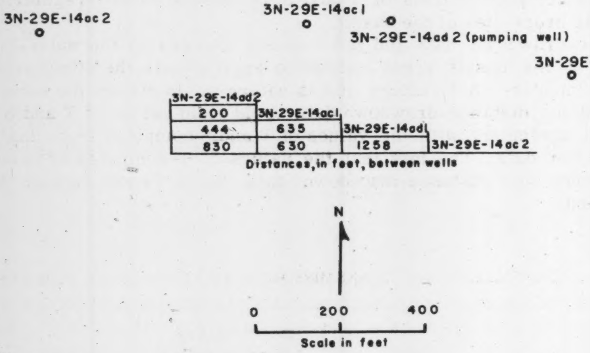
Well No. (1)	Type of data (2)	Coefficient of transmissibility, in gallons per day per foot (3)	Coefficient of storage (4)
6N-31E-13ac2	Time-drawdown	800,000	—
6N-31E-13ac1	Time-drawdown	800,000	0.02
6N-31E-13ca1	Time-drawdown	700,000	0.02
6N-31E-13ac1 and 6N-31E-13ca1	Distance-drawdown	900,000	0.01
	Average (rounded)	800,000	0.02

It is unfortunate that at least three wells at different distances from the pumped well were not available for the test, because distance-drawdown curves based on at least three points are desirable. It is seldom, however, that more than one observation well is available for measurement during a test on the Snake River plain.

Cones of influence do encounter boundaries during aquifer tests in basalt in some places. The effects on water levels of the presence of boundaries are illustrated in the results of an aquifer test made in the southern part of the NRTS area.

An aquifer test (test 2) was made on June 7-11, 1957, with the wells shown in Fig. 6. The effects of the discharge of well 3N-29E-14ad2 were measured in wells 3N-29E-14ac1 and 3N-29E-14ad1. Generalized graphical logs of the wells are given in Fig. 7. Pumping was started at 10:50 a.m. on June 7 and was continued at a constant rate of 4,350 gpm for a period of 48 hr. Pumping was stopped at 10:50 a.m. on June 9 and the water level in the aquifer was allowed to recover. Fig. 8 shows the effects on water levels caused by operation of the pump.

Drawdowns in the observation wells adjusted for atmospheric-pressure changes were plotted against time on logarithmic paper. As an example, the time-drawdown data for well 3N-29E-14ad1 are shown in Fig. 9. For the example given in Fig. 9 $W(u) = 1.0$, $u = 0.1$, $s = 0.029$ ft, $t = 9.8$ min, $Q = 4{,}350$ gpm and $r = 444$ ft. Using Eq. 1



	3N-29E-14ad2	3N-29E-14ac1	3N-29E-14ad1
3N-29E-14ac1	200		
3N-29E-14ad1	444	635	
3N-29E-14ac2	830	630	1258

Distances, in feet, between wells

FIG. 6.—MAP SHOWING LOCATIONS OF WELLS USED IN TEST 2

$$T = 17{,}000{,}000 \text{ gpd per ft}$$

and inserting these values in Eq. 2 yields

$$S = 0.032$$

Using the equation

$$r_i = r_p \sqrt{\frac{t_i}{t_p}} \quad \dots\dots\dots\dots\dots\dots\dots\dots \quad (3)$$

for the data given in Fig. 9, yields

$$r_{i_1} = 444 \sqrt{\frac{900}{12.6}} = 3{,}700 \text{ ft}$$

and

$$r_{i_2} = 444 \sqrt{\frac{4{,}000}{12.6}} = 7{,}900 \text{ ft}$$

After pumping started, the water level in the observation well declined for a time under the influence of the pumped well only. After about 100 min of pumping, the rate of drawdown increased under the influence of an impermeable boundary. The rate of drawdown increased again after about 300 min of pumping, as the effects of a second impermeable boundary reached the observation well. Thus, by the end of the test the decline of the water level in

well 3N-29E-14ad1 was influenced by the pumped well and by two impermeable boundaries. The time-drawdown data indicate that changes in the water-bearing properties of the basalt great enough to approximate the effect of impermeable boundaries occur in two different directions and at relatively short distances from the pumped well.

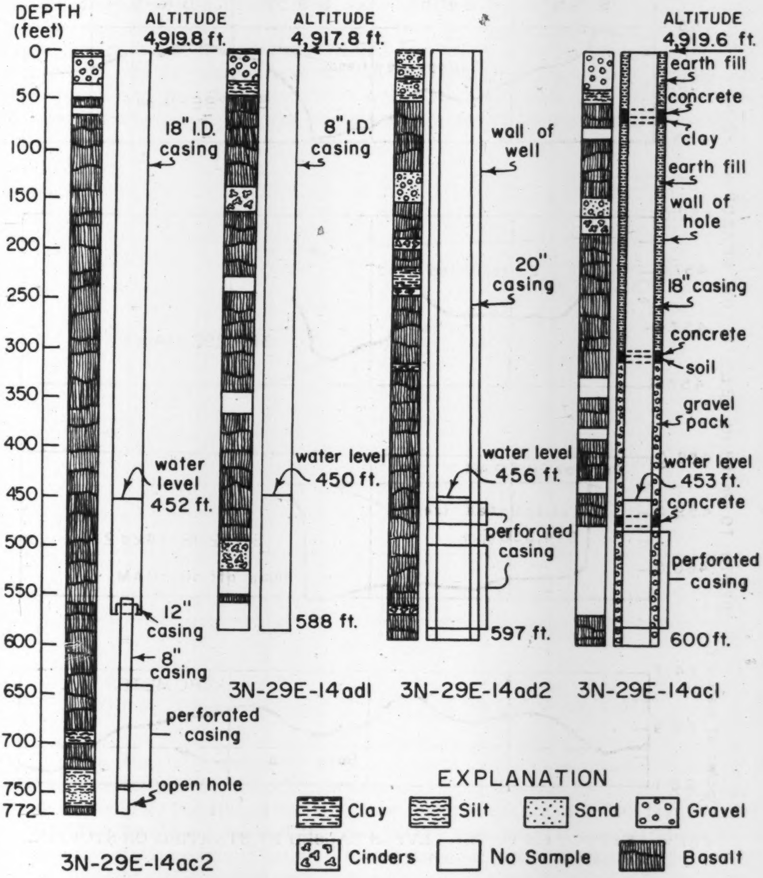

DEPTH (feet)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
772
ALTITUDE 4,919.8 ft.
18" I.D. casing
water level 452 ft.
12" casing
8" casing
perforated casing
open hole
3N-29E-14ac2
ALTITUDE 4,917.8 ft.
8" I.D. casing
water level 450 ft.
588 ft.
3N-29E-14ad1
wall of well
20" casing
water level 456 ft.
perforated casing
597 ft.
3N-29E-14ad2
ALTITUDE 4,919.6 ft.
earth fill
concrete
clay
earth fill
wall of hole
18" casing
concrete
soil
gravel pack
water level 453 ft.
concrete
perforated casing
600 ft.
3N-29E-14ac1
EXPLANATION
Clay
Silt
Sand
Gravel
Cinders
No Sample
Basalt

FIG. 7.—GENERALIZED GRAPHIC LOGS OF WELLS USED IN TEST 2

The nonequilibrium type curve was matched to the portion of the time-drawdown data unaffected by the boundaries and match-point coordinates were used to compute the coefficients of transmissibility and storage of the aquifer. The time-drawdown data for the first 20 min of pumping are interpreted as

departing from the type curve because of the differences between actual field conditions and the ideal conditions assumed in the derivation of the nonequilibrium formula. Thus, the curves were matched for the period between 20 min and 100 min of pumping.

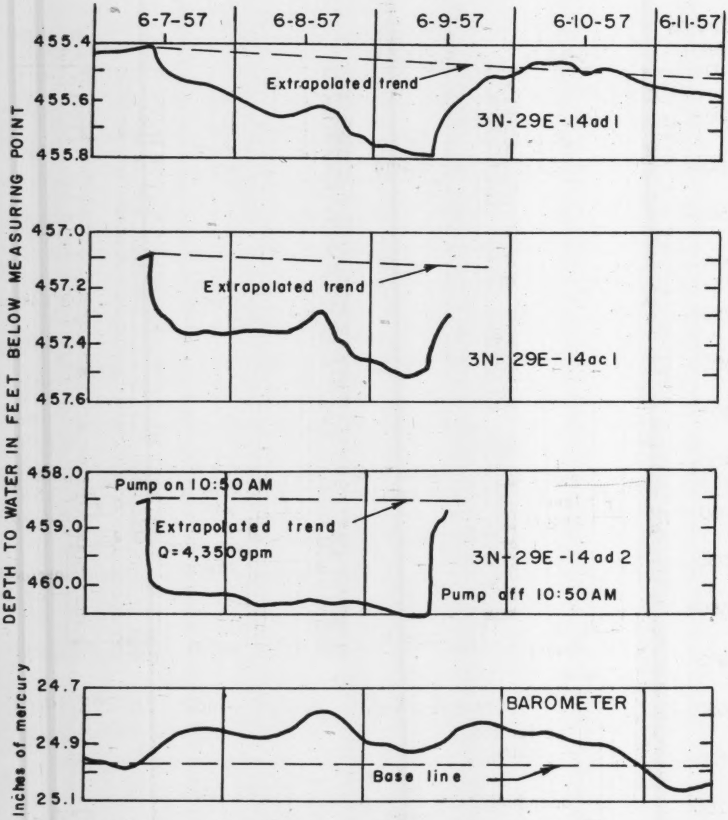


FIG. 8.—EFFECT ON WATER LEVELS CAUSED BY STARTING OR STOPPING PUMP IN WELL 3N-29E-14ad2

It should be noted that the initial period during which the cone of depression adjusted to the nonuniform water-bearing characteristics of the basalt (20 min) is interpreted as being much shorter during this test than it was during the test described previously (90 min). According to the driller's logs, the water table in the vicinity of the site of test 2 occurs in a zone of soft, broken, porous lava and cinders, whereas the water table in the vicinity of the site of test 1 occurs in slightly- to moderately-jointed basalt. The characteristics of the water-bearing openings of the basalt at the site of test 2 are more like those assumed in the derivation of the nonequilibrium formula than are those at the site of test 1. It is, therefore, logical to assume a shorter adjustment period for test 2 than for test 1. In addition, the observation wells of test 2 are closer to the pumped well than those of test 1.

The effects of the boundaries were analyzed by means of the image-well theory and the law of times.[5] The type curve was matched to the time-drawdown data for the portions of this test affected by the image wells. The divergence of the time-drawdown curve from the type-curve traces was determined and the distances from wells 3N-29E-14ad1 and 3N-29E-14ac1 to the two image wells was computed, by use of the method illustrated in Fig. 9. The distances between the image wells and the observation wells, as well as the values of T and S determined from the results of the test, are listed in Table 2. Possible errors in tape measurements of water level made in well 3N-29E-14ac1 toward the end of the test made it impossible to determine the distance from that well to the farthest image well.

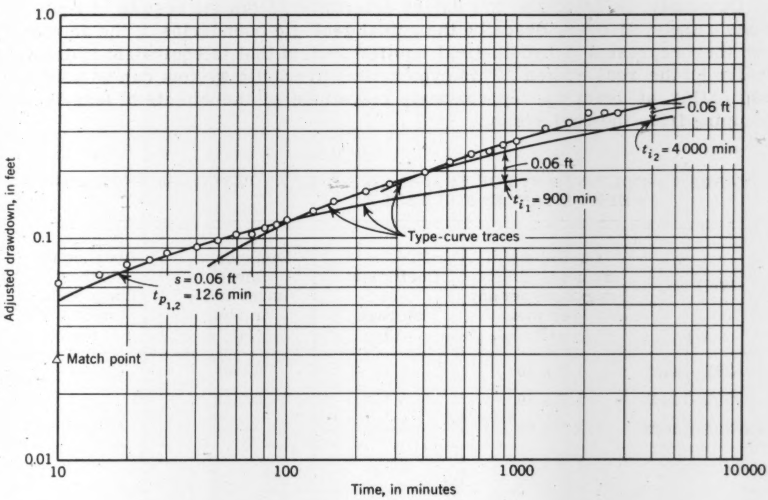


FIG. 9.—TIME-DRAWDOWN GRAPH FOR WELL 3N-29E-14ad1

The impermeable boundaries in the basalt at the site of test 2 cannot be located exactly because only two observation wells were available during the test, and the data for one of the observation wells are poor owing to erratic tape measurements. However, the results of the test suggest that one of the boundaries is about 1,600 ft west of the pumped well, because two arcs associated with the closest image well nearly intersect west of and about 3,200 ft from the pumped well. The data indicate also that the distance from the pumped well to the other boundary is roughly 4,000 ft. The orientations of the two boundaries cannot be determined.

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[5] "Hydrology," by C. O. Wisler, and E. F. Brater, John Wiley and Sons, Inc., New York, 1951; Chapter 7, "Groundwater," by J. G. Ferris, p. 247.

The poor performance of production well 3N-29E-14ac2, about 830 ft west of well 3N-29E-14ad2, supports the conclusion that a boundary does exist west of the pumped well. Well 3N-29E-14ac2 was originally drilled to 601 ft, about 145 ft below the water table, and had a specific capacity of 12 gpm per ft of drawdown for a pumping period of 24 hr. The well was deepened to 772 ft and tested again. The specific capacity was not increased. For comparison, the specific capacity of well 3N-29E-14ad2 for a pumping period of 24 hr was 2,175 gpm per ft.

Obviously the idealized boundaries determined from the results of aquifer tests rarely, if ever, describe the actual geologic boundaries of the aquifer, but they represent a hypothetical aquifer system that is equivalent hydraulically to the real system. This hypothetical hydraulic system can be used in estimating at least the approximate magnitude of the effects of long-term pumping from the real system.

TABLE 2.—COEFFICIENTS OF TRANSMISSIBILITY AND STORAGE AND IMAGE-WELL DISTANCES FOR TEST 2

Well No. (1)	Coefficient of transmissibility, in gallons per day per foot (2)	Coefficient of storage (3)	Distance to closest image well, in feet (4)	Distance to farthest image well, in feet (5)
3N-29E-14ac1	1.7×10^7	0.08	3,000	——
3N-29E-14ad1	1.7×10^7	0.03	3,700	7,900
3N-29E-14ad2	2.0×10^7	——	——	——
Average	1.8×10^7	0.06		

A summary of the aquifer-test data collected to data on the Snake River plain and the coefficients of transmissibility and storage of the Snake River basalt computed therefrom are given in Table 3. Few data are available for areas on the Snake River plain outside the NRTS. It should be emphasized that the values listed in the table are provisional and subject to revision as further field data are obtained and more is learned about the occurrence and movement of water in the basalt.

Values of the coefficient of transmissibility of the basalt aquifer given in Table 3 range between 1.6×10^5 and 1.8×10^7 gpd per ft. The average value of T is 4.2×10^6 gpd per ft. For the wells tested the average penetration below the regional water table is about 130 ft. The aquifer ranks as one of the most productive in the United States.

For many of the tests made at the NRTS no observation wells were available and measurements of drawdown were obtained only in the pumped wells.

The coefficient of storage of an aquifer cannot be determined by use of data collected in a pumped well unless the effective radius of the well and the entrance losses of head are known. Satisfactory approximations of these quantities can be obtained only under favorable conditions.

The coefficients of storage computed from test data are all indicative of water-table conditions, as they range from 0.02 to 0.06. The coefficient of storage of the Snake River basalt is comparatively low for a water-table aquifer, although reasonably high for one in consolidated rock. Storage coefficients exceeding 0.20 are common for unconfined sand and gravel aquifers.

TABLE 3.—RESULTS OF AQUIFER TESTS IN THE SNAKE RIVER BASALT

Location of test (1)	Wells used in test (2)	Coefficient of transmissibility, in gallons per day per foot (3)	Coefficient of storage (4)
Minidoka	8S-24E-8ad1 8S-24E-8ad2	4.8 x 10^6	0.04
Mud Lake basin	5 wells in Owaley Canal Company well field north of North Lake in Jefferson County	1.1 x 10^7	0.06
NRTS area	6N-31E-13ac2 6N-31E-13ac1 6N-31E-13ca1	8.0 x 10^5	0.02
	3N-29E-14ac1 3N-29E-14ad1 3N-29E-14ad2	1.8 x 10^7	0.06
	3N-29E-24ad1 3N-30E-19bc1 3N-30E-19cb1	3.3 x 10^6	0.06
	5N-31E-10cd1	5.7 x 10^5	—
NRTS area	5N-35E-13db1	7.5 x 10^5	—
	4N-30E-7ad1	1.7 x 10^6	—
	4N-30E-30aa1	1.5 x 10^6	—
	4N-30E-30ad1	3.7 x 10^6	—
	2N-29E-1db1	1.6 x 10^5	—

The coefficients of storage computed from the test data pertain, in large part, to the portion of the aquifer through which the water table moved during the tests and may not be representative of the basalt as a whole. Further, because of slow draining, longer pumping tests might yield coefficients of storage even larger than those computed.

FACTORS COMPLICATING ANALYSIS OF AQUIFER-TEST DATA

The drawdowns in observation wells during aquifer tests in the Snake River basalt are small, generally less than 0.5 ft, for practical pumping rates. Even though a steel tape is used, unavoidable small errors in water-level measurements are often made because the water table lies rather far below the land surface (the depth to water was about 215 ft in one of the tests described in detail, and 450 ft in the other). Also, precise records of water-level fluctuations are difficult to obtain with float-actuated recording gages because proper balance of floats and counterweights is hard to attain at the depths involved. As a result, floats tend to hang up on the walls of wells and produce erratic water-level records. These unavoidable small errors in water-level measurements amount to an appreciable percentage of the total drawdown, especially during the latter part of tests when the rate of drawdown is small. The data are, therefore, commonly scattered.

The water levels in wells in the Snake River basalt are affected to a great degree by atmospheric-pressure changes. At times, small drawdowns in a well are completely masked by water-level fluctuations caused by atmospheric-pressure changes. Consequently, adjustments of drawdown data for atmospheric-pressure changes are often equal to or greater than the corrected drawdowns themselves.

Adjustments of drawdown data for atmospheric-pressure changes are based on a constant barometric efficiency (the ratio of the change in the water level in a well to the change in atmospheric pressure, expressed in feet of water). For diurnal atmospheric-pressure fluctuations in the summer there is essentially very little difference in the barometric efficiency of a well during periods of increasing and decreasing atmospheric pressure. However, during the winter when pressure changes are large and often erratic, the barometric efficiency of a well is not the same at all times. As a result, water-level data, adjusted for changes in atmospheric pressure based on a constant value of barometric efficiency, are often irregular, which further complicates the analysis of the data.

SPECIFIC-CAPACITY DATA

Specific-capacity data obtained during well-acceptance tests can be used to augment aquifer-test data in determining the approximate range of the coefficient of transmissibility.[6] The specific capacity of a well may be defined as the yield of the well per unit of drawdown and is generally expressed in gallons per minute per foot. A high specific capacity indicates that the aquifer supplying the well has a high coefficient of transmissibility.

Factors other than the coefficient of transmissibility, however, bear on the specific capacity of a well. The coefficient of storage, the depth of pene-

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[6] "Estimating Transmissibility from Specific Capacity," by C. V. Theis, et al., Open-File Report, U.S.G.S., 1954.

TABLE 4.—SPECIFIC-CAPACITY DATA FOR WELLS IN THE SNAKE RIVER BASALT

Location of wells (1)	No. of wells for which data are available (2)	Average depth of well, in feet (3)	Average penetration of well below water table, in feet (4)	Average diameter of casing in inches (5)	Average length of test, in hours (6)	Range of specific capacity, in gallons per minute per foot (7)	Average specific capacity, in gallons per minute per foot (8)	Average pumping rate during test, in gallons per minute (9)
1. Jefferson County	55	200	72	16	16	33,14,000	3,200	2,400
2. U. S. Bureau of Reclamation, Minidoka Project, North Side Pumping Division.								
Group 1	15	266	83	21	——	110- 9,000	2,300	2,9000
Minidoka, 2	21	264	94	24	2 1/2	110-20,000	4,500	2,500
Jerome, and 3	29	282	91	21	3	200- 6,000	1,700	2,200
Lincoln 4	31	276	88	21	3	59-14,000	1,800 2400	1,800
Counties 5	22	308	83	21	4 1/2	23-17,000	1,800	1,700
6	18	402	112	21	2 1/2	63- 3,000	800	1,500
7	30	271	71	21	3 1/2	150-22,000	3,900	1,800
3. Bingham County	15	252	71	18	8	16- 7,700	1,800	1,900
4. NRTS Butte County	17	590	200	16	24	12- 4,000	1,100	1,300
5. Gooding County	9	120	60	12	2	300- 2,700	900	1,000
6. Jerome County	6	362	77	16	3	6- 1,600	900	1,700
7. Mud Lake basin (shallow aquifer) Jefferson County	9	78	57	24	2	260- 2,700	800	4,100
8. Bonneville County	6	207	88	18	4	33- 1,000	500	1,800
9. Blaine County	2	226	179	16	2	37- 110	70	1,400
10. Camas County	3	210	77	16	6	30- 110	60	1,200

tration of the well below the regional water table, the size of the well, the entrance losses, and the duration of the pumping period also affect the specific capacity. However, the approximate relation between the coefficient of transmissibility and the specific capacity of a well can be determined from the results of aquifer tests. This relationship can then be used to obtain a rough estimate of the coefficient of transmissibility of the basalt in the vicinity of a well in an area where aquifer-test data are not available but specific-capacity data are.

Table 4 summarizes data obtained on the specific capacities of production wells in the Snake River basalt.

The data indicate that, for the Snake River plain as a whole, the average yield of the basalt to a 16-in. well during a well-acceptance test of about 8 hr is about 2,100 gpm per ft of drawdown. The average depth of wells below the land surface and average penetration below the regional water table are 290 ft and 100 ft, respectively. Although specific capacities ranged from 6 gpm per ft to 22,000 gpm per ft, the common range in specific capacity was from 60 gpm per ft to 3,200 gpm per ft.

Judging from the common range and average of the specific capacities of wells, the coefficient of transmissibility of the Snake River basalt ranges between 1×10^5 gpd and 7×10^6 gpd per ft and averages about 4×10^6 gpd per ft. The wells for which specific capacities and aquifer-test data are available penetrate the basalt only partially. Massive layers of relatively impermeable basalt commonly isolate the lower part of the aquifer from the material penetrated by wells. It is probable, therefore, that the coefficient of transmissibility of the entire thickness of the Snake River basalt greatly exceeds the value determined from data on partially penetrating wells.

The yield of wells in basalt sometimes greatly increases with small increases in the depth of penetration of the well below the regional water table as highly permeable zones are encountered by a well. As an example, well 4N-30E-30aa1 on NRTS was drilled to a depth of 483 ft and showed a specific capacity at that depth of 16 gpm per ft. Later the well was deepened to 535 ft and its specific capacity increased to 2,800 gpm per ft. The yield of a well does not always increase with depth. For example, the yield of a well 3N-29E-14ac2 on the NRTS was not appreciably increased by deepening the well from 601 ft to 772 ft.

ACKNOWLEDGMENTS

The field and office work of many members of the USGS in addition to the writers is embodied in the results of this paper. Several participated in the writing of earlier special reports that have been used as reference material. Grateful acknowledgment is made, therefore, to the following members of the USGS: J. T. Barraclough, E. G. Crosthwaite, Morris Deutsch, F. E. Fennerty, J. R. Jones, I. S. McQueen, R. L. Nace, R. C. Scott, Rex O. Smith, C. V. Theis, W. I. Travis, M. ASCE, P. T. Voegeli, and E. H. Walker. The writers are also indebted for background material used in this paper to Lynn Crandall, F. ASCE, Henry Gannett, G. R. Mansfield, A. C. Peale, I. C. Russell, H. T. Stearns, W. G. Steward, and Orestes St. John. Acknowledgment is made to officials of the AEC, United States Bureau of Reclamation, and State of Idaho Department of Reclamation, who were most cooperative and helpful in making

data available on well-acceptance tests and in providing the facilities required to make aquifer tests.

CONCLUSIONS

The characteristics of the water-bearing openings in the Snake River basalt are not exactly as assumed in ground-water formulas. The basalt aquifer is heterogeneous and anistropic. However, despite the differences between assumed and actual field conditions, meaningful values for hydraulic properties can be computed from the results of aquifer tests. The inability of the basalt aquifer to respond to pumping instantaneously as would a homogeneous, isotropic, and perfectly elastic artesian aquifer must be taken into consideration in applying ground-water formulas to aquifer test data.

Little is known about the coefficient of storage of the Snake River basalt because during many aquifer tests observation wells were not available. More complete aquifer tests utilizing two more observation wells at different distances from the pumped well should be made in the future to determine the regional variation in the coefficient of storage.

The role of individual flow-contact zones penetrated by wells as contributors of water should be appraised.

The Minidoka North Side Pumping Division project appears to offer an unusual opportunity for a detailed quantitative study of the effects of a heavy concentration of pumping. Records of past pumpage and water levels would provide important data on the hydraulic properties of the basalt aquifer.

DISCUSSION

M. MAASLAND,[7] M. ASCE.—Walton and Stewart have presented an interesting analysis of pumping tests at two sites and a sound discussion of the applicability and limitations of the theory of non-stationary flow of groundwater into wells. The authors state that the transmissibility of the Snake River basalt ranges from 1×10^5 gpd per ft to 1.8×10^7 gpd per ft and averages about 4×10^6 gpd per ft. Close reading of the article seems to indicate that the transmissibility effectively varies from zero to an as yet unknown upper limit. The authors are very careful in pointing out the limitations of the analyses and indicate that none of the wells tested had complete penetration. In some areas the transmissibility may be greatly in excess of the values reported.

Perhaps the authors would explain their reasons for making such detailed, expensive pumping tests which, in themselves, are no more than "spot" measurements if the great areal extent of the Snake River basalt is considered. As a matter of fact, the writer wonders whether it would not have been advantageous to omit detailed tests in favor of collecting information concerning the discharge, drawdown, and general well performance over a larger area. Conclusions might be drawn from such information by the methods used by Walton and Stewart for analyzing specific-capacity data.

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[7] Civ. Engr., TAMS, Adana, Turkey; formerly, Hydr. Engr., U. S. Bur. of Reclamation, McCook, Nebr.

It is stated that the area of basalt sampled has a diameter of roughly eight miles. This does not seem consistent with the statement in the preceding paragraph that there was no significant drawdown in a test well located 5,450 ft from the pump well. Justification for this observation is not clear.

W. C. WALTON,[8] M. ASCE.—Maasland questioned the value of pumping tests by stating that pumping tests "are no more than 'spot' measurements." The pumping tests discussed in the paper sampled areas of basalt having diameters of several miles. It would seem to the writer that the "spot" is rather large. The value of the pumping test in evaluating the occurrence and availability of ground water has been discussed in numerous reports and will not be repeated here.

All data concerning the discharge, drawdown, and general performance over the entire Snake River plain, available at the time of the writing of the report (September, 1957) were included in the report. Since that time, additional data have become available largely as the result of pumping tests made by the United States Geological Survey (USGS) in cooperation with the United States Bureau of Reclamation (USBR). It is probable that our knowledge concerning the hydraulic properties of the Snake River basalt has greatly increased over the last three years.

Maasland implies that two paragraphs under the section heading "Aquifer-Test Data" are inconsistent. The statement was made that the drawdown in well 6N-31E-12ac1 (5,450 ft from the pumped well) was in the magnitude of 0.1 ft at the end of the test. A drawdown of 0.1 ft is not insignificant as indicated by Maasland but is appreciable and is in accord with the distance-drawdown graph given in Fig. 5. Fig. 5 was used to determine the area of basalt sampled during the test. Therefore, the noted material presented is consistent.

Maasland is correct in stating that the coefficient of transmissibility has an unknown upper limit. A large lava tube must have an unusually large coefficient of transmissibility.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 3238

LEGAL ASPECTS OF GROUND WATER UTILIZATION

By Robert O. Thomas,[1] F. ASCE

With Discussion by Messrs. Raphael G. Kazmann; Frederick L. Hotes; Paul Baumann; and Robert O. Thomas

SYNOPSIS

The historical basis of water rights doctrines; their application to rights to the use of ground water; general considerations affecting the regulation of rights; and various aspects of ground water utilization, including problems of recharge and water demand are presented herein. In conclusion a presentation is made of some of the possible legal problem areas that may arise in planning and administering future ground water operations.

INTRODUCTION

As population, the general economy, and the standard of living continue to increase, the demands on the available water supply also increase, often in geometrical proportion. The attention of engineers and others immediately concerned with the provision of adequate water supplies is turning, with increasing momentum, to the possibilities inherent in the planned utilization of

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Note.—Published essentially as printed here, in December, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2283. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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ground water storage capacity. An attempt will be made to point out some of the more readily apparent legal aspects of such utilization.

HISTORICAL BASIS OF WATER RIGHTS DOCTRINES

Law originates in the efforts of a group to evolve rules governing the actions and relations of the members of the group between themselves as individuals, between members of the group and the group itself, and between members of more than one group. Much of our law pertains to the rights and duties of those who control or possess physical objects. Thus, we have a tremendous body of law governing such material objects as land, buildings, machines, money, personal property, and water.

Water, however, is peculiar in that it constitutes both a continuously renewable natural resource and a resource that is subject to dissipation with time. Where as it would be physically possible to capture and segregate a given volume of water, it would be of slight, if any, value to the owner, as in the course of time it would disappear due to evaporation and, possibly, seepage. From the earliest times, it has been recognized that the value of water lies in its use -- for drinking and washing, for dilution of wastes, for agriculture, navigation, recreation, and countless other purposes. The law concerning the possession and control of water relates principally to the rights and duties in the use of water, rather than to the physical control of the corpus of the water.

Water rights, as we use the term, are essentially rights to the use of water and as such, are recognized by law. Such rights, whether originating under riparian or appropriation doctrines, are rights in real property. Water rights, under both the riparian and appropriation doctrines are generally acquired and exercised in accordance with state laws governing water. The United States has, to some extent, recognized the power of each state to adopt its own system of water law. Congress has not established any procedures for the acquisition of such rights from the Federal Government, but, on the contrary, has required certain federal executive agencies to comply with state legislation affecting water rights. This requirement, however, has not been made specifically applicable to all federal agencies.

In the United States, two general systems of water right law are in existence. One system, prevalent in the eastern states, is the riparian system. The other, followed generally in the seventeen arid western states, is known as the "doctrine of prior appropriation." Even in these latter states, however, the riparian doctrine, or system, is recognized, although to a considerably lesser extent. In some states it has been abolished by legislation.

The major distinctions between the riparian and appropriative doctrines arise from the conditions created by the contrasts in climate in the broad regions where they were first developed and applied. The humid eastern portion of United States generally has an annual average precipitation exceeding 30 in. In the seventeen arid western states, on the other hand, water deficiencies are accepted as normal during most years.

In the humid states, farmers may generally anticipate annual precipitation sufficient for their needs; indeed, there may even be an unused surplus remaining. Land ownership, and an accompanying riparian doctrine, may form logical bases for water rights in surplus water supply regions. Where irrigation, however, is encouraged, or practiced, a comprehensive system of legal rights and duties, as related to water, is generally required. The requirement for municipal and industrial uses also creates problems in developing adequate supplies. This has sometimes necessitated the development of distant surplus supplies as, for instance, the Croton and Catskill water supply systems of the city of New York. The riparian doctrine does not lend itself to such development.

On the other hand, in the arid west the annual rainfall at lower elevations is usually insufficient to supply the needs of developed valley lands. The mountain ranges, however, provide the means of storing the snowpack, and thus make available a source of water for beneficial uses. The locality with a requirement for water, however, is often far removed from the region of its occurrence. Consequently, a system based on appropriation, whereby water may be diverted, placed in temporary or carry-over storage, and conveyed to the place of need, without regard to watershed limits, is a basic necessity.

The riparian system was originally derived, not from the great body of common law as first inherited from England, but from early Roman law and its legal descendant, the Code Napoleon. There are many indications that the early English common law, while incorporating some aspects of what has come to be known as "riparian doctrine," included consideration of the "first in time, first in right" theory, a basic tenet of the doctrine of appropriation. As late as 1831, in the case of Liggins versus Inge, 131 Eng. Rep. R. 263, Chief Justice Tindall, of the Court of Common Pleas, wrote:

> "By the law of England, the person who first appropriates any part of the water flowing through his own land to his own use has the right to the use of so much as he thus appropriates against any other."

In 1918, Samuel C. Wiel,[2] an authority on the development of American water law, said,

> "The word 'riparian,' as the name of a law of watercourses, made (its) first appearance in English reports in 1849 . . . . In the Seventeenth century . . . a pleading of prescription or ancient custom was held to be the proper allegation; the law of watercourses was treated as resting upon prescription or ancient custom . . . (In the Eighteenth and Nineteenth centuries) the tendency was to pass from this to the doctrine of prior possession whether ancient or not -- the doctrine of prior appropriation."

The Institute of Justinian decreed that, by natural law, running water was owned in common by all free men. It naturally followed that an individual was precluded from exercising an exclusive use of the water supply. The Code Napoleon (French Civil Law) permitted a landowner to use water from a stream crossing or bordering his property, provided he restored it to the watercourse at its point of departure from his land. These expressions of the continental civil law are generally credited as the source of the riparian doctrine brought into American law in the nineteenth century by Justice Joseph Story.

The riparian doctrine, as we know it today, was given expression in decisions prepared by Justice Story of the United States Supreme Court and Chancellor James Kent of New York. Justice Story is thought to have used the word "riparian" for the first time in the case of Tyler versus Wilkinson, 4 Mason 397, in 1827. In 1828, Kent's opinion, developing the riparian doctrine, referred to most of the known civil law on the subject, including the Code Napoleon, the Institute of Justinian, and the treatise of Pothier. The riparian principles enunciated by those American justices first appeared to affect English decisions in 1849. In the precedent-setting case, Wood versus Waud, 3 Exch (England) 748 (1849) the English court quoted the American views with approval. Since that time, the case has been used as one of the earliest precedents in applying riparian law. The decision in Wood versus Waud is briefly summarized as follows:

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[2] California Law Review, Article by Samuel C. Wiel, Univ. of California, Berkeley, Calif., Vol. 6, 1918, p. 245.

> "We think . . . that the plaintiffs have received damage in point of law. They had a right to the natural stream flowing through the land, in its natural state, as an incident to the right to the land on which the watercourse flowed; and that right continues, except so far as it may have been derogated from by user or by grant . . . ."

The California Supreme Court, in Lux versus Haggin, 10 Pac 674 (1886), summarized the law and the doctrine prevalent at that time as follows:

> "The nature of the right . . . is usufructuary, and consists not so much in the fluid itself as in its uses . . . every riparian proprietor has a natural and equal right to the use of the water in the stream adjacent to his land, without dimunition or alteration . . . and to have the stream . . . flow as is wont by nature . . . undiminished except by its reasonable consumption by upper proprietors."

Water Rights in the Western States.—Strangely enough, most of the western states, although deriving their background from the early Spanish exploration and settlement, followed early English leadership in the development of their statutory systems of appropriative water rights. The appropriation doctrine in the western states originated in the customs of the early day miners. Because water was a necessary adjunct to the working of claims, miners, as individuals and groups, diverted stream flows by means of dams, ditches, flumes, and other structures for storing and conveying the water to the point of use; often outside the watershed in which it had been diverted. Principles basically applicable to possession of property in general, and to mining development in particular, were applied to waters, such as priority of discovery and use, protection against other claimants, and so on. By these means, the miners provided for the appropriation of water by diverting it from streams and devoting it to use, and for public recognition and protection of the property right that was thus asserted.

The importation of the English common law into the United States legal system, providing case law precedents where statutes were not applicable, resulted in the re-introduction of English precedents based on American cases that were, in turn, based on continental legal codes. This was particularly confusing in states, such as California, where the state constitution provided for the adoption of common law precedents in all cases not covered by state statutes.

The adoption of English common law, as it related to water, placed the existing system of appropriation squarely at issue with the doctrine of riparian rights. The early appropriations of water had been made on what were, at the time, public lands. These appropriations were confirmed by Congress in the Acts of 1866, 1870, and 1877. The question of appropriative rights versus riparian rights appurtenant to private lands was resolved, basically, by the application of rules developed for the protection of possessory rights in property secured by prescription.

Various modifications of the riparian doctrine have been made in court decisions in the several western states. Since 1928, California has limited the riparian owner to a prior and paramount right extending only to the reasonable beneficial use of the water under reasonable methods of diversion—Gin S. Chow versus Santa Barbara 22 P. (2d) (1933), Meridian Limited versus San Francisco, 90 P. (2d) 537 (1939), Peabody versus Vallejo, 40 P. (2d) 486 (1935). In Kansas, riparian owners have the primary right to use all the water they may require for domestic use and for watering livestock. Subsequently, after all other riparian landowners have had the opportunity for similar uses of the available water, they are all equally entitled to an equitable share, for irrigation purposes, of water remaining in the stream. In South Dakota, a riparian owner may exhaust a stream for domestic use and stock watering, otherwise the rights of all riparians for other beneficial uses are equal. In Texas, riparian waters include only the ordinary flow and underflow of the streams, and waters of a stream above the level of highest ordinary flow are regarded as flood waters, to which riparian rights do not attach.

In some of the western states, however, the system of riparian rights has been discarded in its entirety. In these states, the only methods of acquiring a right to the use of water is by appropriation, or by license from the state or both of these methods. In others, the rigidity of the riparian system has been eased to some extent by the application of the concepts of reasonable use, and reasonable methods of diversion of use, to the quantity of water needed to satisfy the requirements of the riparian landowner. These concepts have made possible the appropriation and diversion of surplus waters not required for use on riparian lands. The burden of proof establishing the existence of such surplus, however, remains the responsibility of the appropriator.

Utah has perhaps proceeded further in this direction than other states. For many years, the Utah courts had declared that underground water was owned by the overlying landowner. In 1943, however, the state laws regulating water were amended to provide that underground water was subject to appropriation. The amendment provided that,

> All waters in this state, whether above or under the ground, are hereby declared to be the property of the public, subject to all existing rights to the use thereof.

In a subsequent case, Riordan versus Westwood, 203 P. (2d) 922, this statute was upheld and, in effect, all prior decisions in conflict were held to be based upon erroneous interpretations of law. Those who had previously used ground water had, of course, acquired a prescriptive right to continue such existing use. All other underground water, however, was subject to appropriation under the jurisdiction of the State Engineer. In its decision, the court said,

> "Until 1935, the decisions of this court treated the waters of artesian basins as percolating waters, and as such the ownership went with the owner of the ground where such waters were located. . . . Our concept of what was and what was not percolating waters has changed greatly since that term was used in the early cases . . . in an arid state like ours it is very important that all of the waters be used in the most beneficial and economical way possible and that none shall be wasted . . . The question must be determined on our present standards and concepts and we must treat that question as though our concepts and standards had always been as they are now . . ."

Water Rights in the Eastern States.—In the eastern states, water has not in the past been the key to the existence of a landed economy, as it has in the west. On the contrary, the very profusion of water supplies has been a major problem. In consequence, the principal purposes of water legislation have been the alleviation of flood problems, the provision of drainage, the improvement of navigation, and the maintenance of the quality of surface supplies. In more recent times, ground water development has, in some areas, resulted in the passage of legislation regulating the control and use of such supplies.

The establishment and maintenance of rights in and to the use of water has principally been accomplished by litigation in the courts. The resulting case law has essentially followed the philosophy expressed in the riparian doctrine, frequently modified, with the passage of time, by the concept of reasonable use. Rights to the use of ground water, originally part and parcel of the general property right of the overlying landowner, have gradually become identified with the concepts of reasonable use and the correlative rights doctrine.

Much of the legislation enacted in eastern states has resulted from the need to acknowledge some special situation, rather than from the desirability of establishing general rules to provide for the equitable apportionment of water supplies. Examples of such legislation are the so-called "Mill Acts," stemming from the changes generated by the industrial revolution. These acts generally provided for the erection of dams or other obstructions to stream flow and for the assessment of, and compensation for, damages sustained by riparian proprietors. A number of states, such as, Maryland, Wisconsin, Minnesota, North Carolina, Illinois, New York, New Jersey, and Indiana, have enacted legislation requiring permits to be secured from the state for the use of water in certain circumstances. These laws, however, appear to be quite general in their nature, frequently apply only to water used for certain purposes, and often are limited to specified regions within the state. There seems to be considerable doubt that they are sufficiently comprehensive in philosophy to constitute a reasonable basis for acquiring property rights in the use of the water taken under permit.

In 1954, Indiana, Kentucky, and Virginia adopted legislation declaring that the state possessed a general police power to provide for the protection and utilization of the water resources. These enactments, however, require additional legislation setting forth rules and procedures for implementing the policy declarations. This has been accomplished, in a limited way, by laws regulating certain uses of water by riparian proprietors on surface streams.

Mississippi, in 1956, established a system of water appropriation that, after providing for prior rights, made the use of, and rights in, surface water contingent upon compliance with the law and the administrative provisions regulating the acquisition of water rights.

In the period since World War II, accelerated interest in supplemental irrigation, the recurring danger of drought, and the demands resulting from increasing municipal and industrial use have occasioned a heightened awareness of the philosophy and principles governing the existence of water rights. Many of the eastern states have established commissions for investigation and study of water rights legislation. Proposed legislation has been subject to intensive public debate, and symposia on the subjects of water rights and utilization have been held in numerous localities. One of the most important of these was the conference[3] in Washington, D. C., in 1956, on Water Allocation in the Eastern United States, sponsored by the Conservation Foundation.

A Model Water Use Act has been prepared under the auspices of the University of Michigan Law School, and has been approved by the National Conference of Commissioners of Uniform State Laws. Although the general intent of this legislation is the development of the available water resources in the public interest, the treatment accorded to several important aspects of water use and control in the Model Act appears to be questionable or deficient. Among these are the following:

(1) There does not appear to be any provision for state construction, distribution, or management of water resources.

(2) Existing riparian uses are confirmed, but those that have never been exercised are disallowed. In areas where the legal concept of water rights has been based upon the riparian doctrine, this appears to be fraught with danger.

(3) Except for existing uses of water, that are unlimited in duration, 50 yr is the maximum term for which a right of water use may be secured. The power, or threat of power, to affect the economy of a large area or group of people by withholding their water supply is a potentially dangerous weapon to place in the hands of any administrative agency.

(4) Although the Act enumerates several beneficial uses of water, and implies that they (and others not named) are co-equal in the public interest, it also gives the administrative agency the right to deprive a lower beneficial use of a water supply in order to grant that supply to a higher beneficial use for a limited term. This seems to require that the administrative agency should somehow effect a ranking of beneficial uses in addition to granting unwarranted power of penalization and largesse in dispensing the available water supply.

(5) It is very questionable whether, in the light of recent decisions of the Supreme Court, the Federal Government would be amenable to any provisions requiring limitation of the duration of water use permits and applicable beneficial uses.

(6) It would appear that the Act provides, or must consider, three classes of rights, such as, existing rights, Federal Government rights, and all others. Each of these would apparently be amenable to a different set of jurisdictional and legal principles. The resulting confusion in engineering, operation, and administration of water developments could be catastrophic.

(7) Under the conditions established in the Act, it is doubtful whether financing could be secured for water development projects subject to revocable water use permits.

(8) Last, but not least, in the sense of the subject of this paper, there is no apparent provision made for conjunctive use of ground and surface water supplies, nor for the planned utilization and management of ground water supplies.

Among other provisions of the Model Act that may give rise to serious question is the establishment of a state commission as the responsible agency both for planning the utilization of the water supply and for administering the system of permits for beneficial use. Regardless of the conscious effort to be fair-minded that such a commission would undoubtedly make, there would remain a question regarding the influence of their own planning on their decision to grant or deny a permit for beneficial use. The dual scope of authority and administration would appear to engender basic conflicts.

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[3] "The Law of Water Allocation in the Eastern United States," The Conservation Foundation, edited by David Haber and Stephen W. Bergen, The Ronald Press, New York, N. Y., 1958.

It should be noted that the foregoing criticism of the Model Act is made solely for the purpose of pointing out possible areas of change or revision. The effort and thought that resulted in drafting the Act is recognized as an expression of the enlightened public appreciation of the place of water in our economy. The heightened interest in the law of water throughout the eastern states can only have a beneficial influence on the specific legislation eventually adopted. Thorough discussion of the implications and probable results of alternative courses of action will undoubtedly promote the adoption of a system for administration of the available water supply that will result in action designed to promote best interest of the entire public.

RIGHTS IN GROUND WATER

Water beneath the surface of the ground has, until recent times, generally been classed, legally, as a subject separate and apart from precipitation and surface water supplies. Imperfect understanding of hydrologic processes has often led to such water being considered in the same category as a mineral, with the owner of the surface possessing definite rights to its extraction and use. As a case in point, the court held, in Acton versus Blundell, 12 Mees & W. (England) 324 (1843) that,

> ". . . we think the present case, . . . is not to be governed by the law which applies to rivers and flowing streams, but that it rather falls within that principle which gives to the owner of the soil all that lies beneath the surface; that the land immediately below is his property, whether it is solid rock, or porous ground, or venous earth, or part soil, part water; and that the person who owns the surface may dig therein, and apply all that is there found to his own purposes at his free will and pleasure; and that if, in the exercise of such right, he intercepts or drains off the water collected from underground springs in his neighbor's well, inconvenience to his neighbor . . . cannot become the ground of an action."

In other cases, attempts have been made to prove the existence of underground streams, flowing in "known and definite channels." In California, ground water that can be classified in this manner is legally regarded as identical to water occurring in surface streams, that is, subject to the dual imposition of riparian and appropriative doctrines. In making such a distinction in the classification of underground water, the courts have been at variance with hydrologic principles which recognize that all ground water (with minor exceptions, such as unrecoverable soil moisture, water in closed basins that is dissipated in excessive evapotranspiration, and so on) constitutes a part of the generally available water supply and has a common ultimate disposal with surface streams by means of outflow to the ocean or other large body of water, becoming inseparable from surface flows in the process.

Although the principles of the hydrologic cycle are now recognized in many jurisdictions, together with an understanding of physical laws governing flow, many courts still recognize legal distinctions separating and classifying (a) percolating water, (b) ground water in definite underground streams, and (c) underflow of a surface stream.

Percolating water may be said to be all ground water not included in one of the other classes. This unsatisfactory definition results from the necessity for development of opinion evidence in each case litigated. Various decisions have expressed the concept as,

> "vagrant, wandering drops moving by gravity in any and every direction along the line of least resistance,"
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> "a vast mass of water . . . . always slowly moving downward to the outlet or outlets," and
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> "rain waters which are slowly infiltrating through the soil, or . . . waters seeping through the banks or bed of a stream which have so far left the bed and the other waters as to have lost their character as part of the flow."

The other legal classifications of underground water do not appear to possess a much greater degree of scientific exactitude. A definite underground stream must have characteristics similar to a surface watercourse. In other words, it must have a channel with well-defined limits, a source of supply, a measurable flow in a particular direction, and a substantial existence. These facts are generally established by opinion offered in proof of the legal allegations.

The underflow of a surface stream is perhaps the easiest to define. It consists of the water occupying pore space in the material lying beneath the bed of the surface stream, and supporting the surface stream in its natural state. It is generally required that the surface and subsurface flows be in contact, and that the subsurface water flow in a definite direction corresponding to the surface flow.

Rules of law applied to rights to the use of water differ in accordance with the previous classifications of ground water. The complications often resulting from such arbitrary classifications are apparent in considering the case of a valley stream which is influent at one season of the year and effluent at another season. Yet courts will undertake the herculean task of defining the limits of influence of the stream. This is done despite the fact that the development of the basin and the over-all hydrologic conditions will combine to change the location of the boundary of influence from one day to the next.

Decisions have been rendered which hold that the underflow is a part of the stream and subject to the surface stream rights of use; that ground water flowing in definite underground channels is subject to the principles of riparian doctrine; and that "percolating water" (all other underground water) is subject, variously, to riparian doctrines or correlative right doctrines. Such decisions have often served only to complicate the use and development of ground water.

Several of the western states have abandoned past theories relating to the occurrence and ownership of underground waters and have instituted systems providing for the withdrawal and use of ground water by appropriation. New Mexico has applied this concept to a greater degree than other jurisdictions in the arid region. In that state, the existing water in underground storage has accumulated during the past ages. The average annual recharge is, in many cases, considerably less than the annual use. Consequently, the users, as a group, are actually mining the water and can anticipate a time when total exhaustion will have occurred. In basins where the annual pumpage exceeds the recharge, the State Engineer utilizes available knowledge and procedures to estimate the total quantity of recoverable water in storage. An annual pumpage quota, of sufficient magnitude to deplete the basin over a period of 40 yr, is then determined. Permits to appropriate ground water are issued to prospective pumpers until the annual quota is attained, after which time no additional applications are considered. This raises an interesting question: just what do the people of New Mexico (irrigators, municipalities, and other water using activities) expect to do after exhausting their water supply?

The principle of absolute ownership of percolating water has been modified by numerous decisions in our courts. The modification appears to have had its origin in decisions emanating from eastern jurisdictions—Bassett versus Salisbury Manufacturing Company, 43 N. H. 569 (1862) and Forbell versus City of New York, 164 N. Y. 522 (1900). The rule requiring reasonable use in conformity with the rights and requirements of others, established in these cases, later became known as the American Doctrine of Reasonable Use. The leading California case, Katz versus Walkinshaw, 74 P. 766 (1903), applied this rule to California conditions. The California doctrine of correlative rights founded in this decision states, in effect, that owners of land overlying a common water supply have mutual rights to the reasonable beneficial use of the water on the overlying land. The rights of each stud landowner are held to be correlative with the rights of all other overlying owners, and should the supply be insufficient, the available water may be apportioned, by court decree, among all owners making use of the water. Surplus waters, not needed to meet the requirements of overlying lands, may be taken for use in other areas.

The doctrine of correlative rights has been confirmed and expanded in many subsequent decisions of the courts. An early summarization of this doctrine is found in Montecito Vallye Water Company versus Santa Barbara, 77 P. 1113 (1904), as follows:

> "The main question which this court was called upon to consider, and did consider and decide, was whether the common-law doctrine of absolute ownership in percolating water, the cujus solum doctrine, was or was not, under the peculiar conditions existing in this state, subject to the just limitation under the doctrine of sic utere tuo, and this court recognizing the inevitable injury that must be worked to private interests whichever rule should be held to apply, after much deliberation decided that however differently the rule might be declared in states and countries well and regularly supplied by rainfall, in this state, with its great arid stretches, its seasons of drought, and its irregular meteoric water supply, percolating waters, when circumstances of hardship or injury should be presented in some particular case, must be held under the rule and doctrine of sic utere."

In most of the eastern states, the overlying landowner has usually been considered to possess unlimited rights of use in the ground water beneath his land. In a few states, notably Florida, Indiana, Maryland, Minnesota, New Jersey, New York, Tennessee, and Wisconsin, some legislative regulation of ground water use has been attempted. This has usually been brought about because of the necessity to control some specific use or to ensure the utility of a specific ground water basin, such as that underlying Long Island. The Minnesota legislation is comprehensive, requiring permits for any diversion or use of water, whether from surface or underground sources. However, few criteria for permits are given in the law, and the administration of the law has given it the effect of a registration rather than a regulatory statute.

Case law that developed as a result of water rights contests in the east has been rather sparse, and many of the cases are quite old. The American Rule of Reasonable Use has frequently been looked upon with some favor, although there are many holdings to the effect that the overlying owner cannot, in the absence of legislation, be stopped from damaging his neighbors by unreasonable development. There have been few, if any, cases wherein the western doctrine of correlative rights has been applied.

A few eastern states have recognized that the heavy demands of industry and large urban populations are depleting the underground basins faster than they are being replenished. In general, the legislation has been directed toward limiting withdrawals from new wells, without provision for ground water reservoir operation or management. Such regulation merely delays the time of depletion or exhaustion of available supplies. Some legislation is only for the purpose of requiring recirculation of air conditioning withdrawals in order to prevent undue depletion. Some of the recent proposals for legislation in the eastern states have been quite comprehensive in scope, although there is no apparent unamity of thinking, particularly as regards rights in ground water and utilization of underground basins.

The "Raymond Basin" Case.—An important extension of the doctrine of correlative rights, applied in the California case of Pasadena versus Alhambra, 207 P. (2d) 17 (1949), popularly termed the "Raymond Basin" case, was the concept of mutual prescription by and between all persons or agencies pumping from the underground basin.

When the available supply in a surface stream is, for all practical purposes, exhausted each year through application of the waters to beneficial use by riparian proprietors, or appropriators, or both, the fact is self-evident to all. This is not true, however, when dealing with water withdrawn from underground storage. In the latter situation there is, in addition to the annual accretion, a quantity of water in storage that, in some cases, may be drawn on or mined for a number of years before the effects of withdrawal in excess of the safe yield become evident. The excess withdrawal, however, cannot be replaced unless pumpage is reduced to a quantity considerably less than the safe yield. At the same time, a considerable economy has usually developed on the lands, either overlying or distant, served by the pumped ground water.

In its decision, the court pointed out that the actual interference or invasion of each user's right occurred at the moment the safe yield of the basin was first exceeded. However, during the five year prescriptive period immediately following the first occurrence of overdraft, the earlier pumpers could have protected their rights by proceedings, injunctive or otherwise, against those responsible for the excess pumpage. After the completion of the prescriptive period, however, those causing the excess pumpage will have secured a prescriptive right against all other rights in the basin. At the same time, however, the original, or earliest pumpers, who first held overlying or appropriative (gained through use) rights had also secured a prescriptive right against those responsible for the pumpage in excess of safe yield. To this peculiar situation, the court applied the term, "mutual prescription."

In the Raymond Basin case, the court concluded that each water user had gained and held prescriptive rights against all others equal to the amount of water he had used continuously for a period of 5 yr, and which he had not subsequently failed to use for a similar period. Because the total of these rights exceeded the safe yield of the basin, each user was enjoined from pumping more than his proportionate share of the safe yield, based on the ratio of his rights to the total of all rights in the basin, the reduction in each case amounting to about one-third of the total pumpage. A supplemental supply of water must, of course, be available to the area, otherwise the economy of the region would be adversely affected. In this case, water was available through the facilities of the Metropolitan Water District of Southern California.

REGULATION OF RIGHTS TO THE USE OF WATER

From the foregoing brief review of the historical development of rights to the use of surface and ground waters, it may be concluded that a water right, whether riparian or appropriative, is a species of property. In many jurisdictions, however, the rights to the use of water have been circumscribed to some degree, ranging from total public ownership of all water to restrictions governing the reasonableness of use or method of use.

However, a distinction exists between the adjustment, in specific cases, of relative rights as between individuals, and general regulation of rights for the common benefit. In the first instance, much of the case law pertaining to adjudicated rights is limited by the peculiar circumstances and conditions that led to the litigation. Although various decisions are relied on as guides, their applicability must, in each instance, be determined by the courts.

On the other hand, regulation of rights in certain situations and for specific purposes, when related to the general good, has usually been upheld. When a common restriction is placed on the exercise of a right, the relative rights as between individuals, remain essentially as they were before the restriction. This concept becomes directly applicable to the governmental legislation and regulation that will generally be required in order to permit the planned utilization of ground water basins for the conservation, storage, and distribution of the common water supply.

In those states where rights may be secured by appropriation, the applicable regulations generally provide for license or qualified grant by the state, limitation of quantity to that beneficially used, restrictions controlling type or place of use, and provision for reversion of the right in the event of non-use.

In states where the riparian system is the principal, or only, system recognized, statues and court decisions have often required that the use of water be subject to the test of reasonableness. In those jurisdictions the riparian proprietor has been regarded as the possessor of a primary right of use, but his exercise of this right is limited by the condition that he not interfere unduly with the right of others to water not needed by him.

This being the case, there appears to be no insurmountable reason why restrictions on the use of property cannot be applied to water rights in the same manner, and with the same justification, as restrictions on the use of other species of property. Legal impediments to the right of an owner to make free and untrammelled use of his property, ranging from broad restrictions imposed by the Federal government to municipal ordinances are common in our present-day economy. Examples of such restrictions are the activities of the Interstate Commerce Commission, Federal Trade Commission, state public utility commissions, and similar regulatory agencies. State water pollution and sanitary regulations, municipal ordinances concerning building heights, noise, dedication of certain areas in proposed subdivisions, and similar restrictions are further examples of the power to affect property rights.

If water is endowed with a public interest it must yield to regulation. The right of the state to regulate property devoted to the public use has been conceded since the case of Munn versus Illinois 94 U. S. 113 that established the validity of a statute regulating grain elevators. From this concept, the whole field of public utility regulation has developed. Although a right to the use of water is usually considered to be vested, and consequently not susceptible of abrogation under the guise of general governmental power, it apparently can be qualified by appropriate legislation. As the need for water becomes more critical. means of diversion and use that tend to waste the resource will have to necessarily yield to regulation, even at some expense to the owner of the right. A method of use may be reasonable at one time or place but may become unreasonable as water needs become more critical.

Opposition to regulation will undoubtedly be experienced. It is only human to resent and repulse an attempt to restrict or qualify a right that has previously been enjoyed. In some areas, it will be necessary to base regulation of water rights on the police power of the governmental authority. In 1928, the constitution of the state of California was amended by popular vote. The amendment prohibited unnecessary waste of water and limited riparian rights to reasonable use under reasonable methods of use. Although the measure was passed by general vote of the electorate, and made no mention of the supposed state power supporting the provision, the courts deemed it necessary to resort to the general police power in validating the legislation. A typical case under the amendment was Gin. S. Chow versus City of Santa Barbara, 22 P. (2d) 5 (1933), wherein the court declared:

> "That the constitutional amendment now under consideration is a legitimate exercise of the police power of the state cannot be questioned. It is the highest and most solemn expression of the people of the state in behalf of the general welfare. The present and future well-being and prosperity of the state depend upon the conservation of its lifegiving water."

It appears that, in general, water rights are, or can be made, subject to appropriate regulatory action for the general benefit of the people and the economy. It is probable that the possessor of a water right is primarily interested only in the fact of regulation and its effect upon his exercise of the right. He would, of course, be interested in the determination that a governmental authority has a right, however grounded, to qualify his rights in his property. However, he will generally leave the determination of the basis for that governmental right, under whatever theory or philosophy, whether "police," "commerce," "general welfare," or other expression of power, to the legal profession.

UTILIZATION OF GROUND WATER BASINS

The occurrence and movement of water under the surface of the ground has lost much of its aura of mystery in those sections of the country where the very existence and survival of the economy is dependent on the development and use of ground water. Wells and springs have long been a familiar source of water for domestic and agricultural uses, and industry is turning increasingly to ground waters because of the more constant temperature characteristics and relative purity. During the past half-century, development of ground water has proceeded apace, particularly since the development of electrically operated pumping installations for lifting water from considerable depths.

Use of Ground Water Reservoirs for Conservation Storage.—The maximum development of the available water resources and consequent economic development of the area is realized when the available ground water storage capacity is fully utilized for longtime holdover storage of flood waters. This can be accomplished by operating the ground water reservoir conjunctively with surface reservoirs. The principal difference between the usual surface reservoir operation and conjunctive operation with ground water storage is that the objective is the most economic joint reservoir yield rather than the attainment of a firm surface supply. In conjunctive operation, water is placed in underground storage as rapidly as the recharge capacity of the basin and the annual draft on ground water in storage will allow, drawing the surface reservoir annually to a low water stage. Then resulting available surface reservoir storage capacity will then provide for the conservation of much of the peak flow that was formerly lost.

The surface reservoir serves to intercept and provide temporary storage for large flood flows, where as ground water storage, generally of large total capacity and providing protection from evaporation losses, performs the cyclic storage function. The net result is that a greater portion of the available basin storage capacity is beneficially utilized, with a consequent large increase in the firm seasonal yield of water. It has been found that where surface storage alone may result in a yield of about 50% of the mean annual flow, the same amount of surface storage, operated in conjunction with available ground water storage capacity, would provide an annual yield of about 85% of the mean natural water supply. The cyclic storage capacity required, but not economically available on the surface, is developed in the ground water basins. Planned utilization of ground water storage capacity will thus contribute to the ultimate objective of providing adequate supplies of water to meet the ever-increasing demands of present-day civilization.

Use of Ground Water Reservoirs for Regulation.—In recent years it has been recognized that, in providing for the efficient use of water resources, it is necessary to both increase the quantity available and to facilitate the repeated use of water for many different purposes. The planned utilization of ground water storage capacity will not only serve to augment the surface conservation storage capacity available for both seasonal and cyclic water conservation, but will provide facilities for storage and regulation of water supplies while in transit and at the point of use.

Ground water reservoirs are particularly suitable for the terminal storage of imported water supplies. They can be utilized for the purpose of regulating constant flows from aqueduct facilities to meet the variation in monthly demand in the service area. At the same time, they can be utilized as the distribution facility because, in general, the required water supply may be extracted at or near the point of use. Additionally, they comprise valuable units in a system designed for the recapture, and repeated use, of the same water supply.

Domestic, municipal, and industrial use, in addition to irrigation, maintenance of fish and wildlife, recreation, navigation, generation of hydro-electric power, and disposal of sewage and industrial wastes, among other beneficial uses, creates demands that can only be met through the mechanism of repeated cycles of water use and recovery. Some of these uses will occur in the mountain watershed, prior to the time the water is first taken into conservation storage for general use. Others will occur as the water is in transit from one storage facility to another. Still others will occur in the areas of valley use, where the ability of underground storage to receive, store, and regulate unused water percolating from the surface is particularly important. Additionally, water supplies that have already been subject to some uses, but which are still of suitable quality, may be deliberately percolated in available spreading basins, in order to augment the supply.

Thus, through use of the regulatory capability of the groundwater reservoir, the utility of the available water supply is enhanced. The use, recapture, and re-use of water can continue until such time as it is no longer susceptible of capture, or until the quality has deteriorated to the level where final disposal into the ocean or other large body of water is required.

Water Quality Considerations.—Most water uses require supplies included within certain limiting criteria of quality. Failing this, harmful effects from the use of the water will result. On the other hand, most water uses tend to degrade the quality of the water and thus affect its suitability for subsequent or repeated use. When multiple use of the water supply is the desired result, the maintenance of water quality and prevention of water pollution become vital considerations in the planning and operation of water developments. The right of re-use of a water supply often becomes as important as the right to make the original use. Quality problems have already been widely examined. They are mentioned herein only because there may be potential legal injuries stemming from the utilization of ground water storage.

The quality of ground water is of particular concern to agriculture and industry. In agricultural use, where water is evaporated or transpired, the minerals remain in the soil, being leached into the remainder of the ground water body by ensuing rains or by subsequent irrigation. Unless the salts are sufficiently diluted or removed, large areas of land may be adversely affected and sizeable volumes of valuable groundwater storage capacity may be subject to severe damage. In industrial use, quality requirements for boiler feed water, water used in food processing, and water used in many chemical processes are vital to the successful operation of the business.

The water quality problem, from both a practical and a legal standpoint, is a serious consideration in ground water development and must be carefully considered in basin-wide water resources planning. The extent and type of degradation are dependent on the mineral characteristics of both the water supply and the aquifer, and on the duration and quantity of flow. The resulting quality of the ground water varies with both location and time. Because quality criteria are recognized for most water uses, mineralization and possible degradation has a great influence on the value of the water for beneficial uses.

Recharge of Ground Water Storage Reservoirs.—The attainment of the desirable objective of providing regulation of water supplies in underground basins is, to a considerable extent, contingent on the successful accomplishment of recharge operations. Ground water storage capacity is generally more than adequate for conserving the available water supply, but the recharge capacity through the percolation area is often limited. The transmissibility characteristics of the underground aquifers are an additional factor of prime importance. For these reasons, surface conservation reservoirs must be operated in conjunction with ground water reservoirs, with discharges from the surface reservoirs regulated in accordance with the percolation, transmission, and storage capabilities of the underground basin.

The criteria controlling recharge operations are based on hydrologic principles. The procedures of utilizing such principles in administrative management are complex and difficult. A comprehensive plan for the management and control of water resources should provide for continuity of engineering investigation and the development of adequate hydrologic, geologic, and geographic data; the creation of agencies for management of the coordinated operations; and for the financial basis to support management programs. In most instances, these requirements cannot be provided in limited areas or under private auspices. The areas and functions to be covered are so extensive and affect such a broad sector of public interest that they appear to be feasible of accomplishment only through official agencies.

LEGAL PROBLEMS IN PLANNING FOR UTILIZATION OF GROUND WATER STORAGE

The existence of present rights to water from ground water basins is an important aspect of the utilization of underground storage. If the attempt is made to coordinate usage of the storage capacity with the present pattern of withdrawals in a ground water basin, there probably will be conflict between the holder of existing rights to pumpage of ground water and the operator of the basin for reservoir purposes.

The operation will entail primary dependence on, and draft from, ground water supplies during drought periods often with extreme lowering of the water table. Conversely, during an ensuing wet period, the quantity placed in storage would raise the water table to a high level. It is probable that individual users will not complain during such periods as the water table is held at high levels and they consequently receive the benefit of lower costs of pumping; however, at such times as the water table is deliberately lowered in order to provide firm supplies during dry periods, the administrative agency responsible for such operation might be the target of lawsuits or injunctions on the part of local water users. Legislation providing for the utilization of underground storage capacity should also provide for appropriate adjustment with the owners of existing rights. Necessary water supplies could then be provided to the overlying service area by the agency operating the basin.

There appear to be two broad types of artificial recharge operations. The first occurs where the safe yield of the basin underlying a given service area is insufficient to provide for the water requirements of the area. The second type contemplates the use of the basin underlying a given area for the temporary holdover, or regulatory storage, of water in transit, or for terminal storage of imported water. There may well be a third group wherein the characteristics of the previous two are mixed in varying degrees.

What, then, are the rights of a landowner in one of these areas? It appears to be a settled principle of law that the owner of land owns all that occupies the space beneath his land. Water, however, has been excepted from this ownership in many jurisdictions through the operation of judgments in the courts. However, the general ability of the landowner to develop, or to sell or lease to others, separate and apart from use of the surficial contours of his property, the rights to the minerals that may be found beneath the surface has been consistently upheld. In some jurisdictions, the right of the land owner to control the use of the air space above his land, and even to prohibit its use by low-flying aircraft, has been upheld to a certain extent. The question arises, does he have similar rights in the interstitial voids which exist between the solid particles of matter beneath his land? There is no question but that the owner must be recompensed if it is desired to store water above the surface of his land. Is he also entitled to some recompense by the agency effecting the same result beneath the surface?

Political and governmental legal principles provide that when the majority of landowners in a given area join together in the formation of a municipal corporation, district, or other governmental agency, all owners within the boundaries of the new agency become subject to the agency's jurisdiction. In so doing, they give up certain rights in the use of their property that were previously held. Examples of such grant of privilege are the right of a municipality to enforce zoning laws, building codes, and other ordinances that take away from the owner a part of that which previously was his recognized privilege. He is also subject, in certain types of districts, to liens against his property by reason of bonds or assessments, even though he personally may not have been in favor of the imposition of such bond or assessment. It seems that the general theory upon which this dimunition of freedom to enjoy real property is based is that the landowner receives certain other benefits, such as police and fire protection, educational services, and other miscellaneous services rendered by the governmental agency for the benefit of all. Can this theory be extended to cover the use of the storage capacity in the underground basin by an agency that is organized for that purpose?

Use of the storage capacity to augment the supply available to the overlying landowner would probably be regarded as a benefit for which he gave up a part of his right of ownership of all beneath the surface of his land. However, if the use of the storage capacity beneath his land was for the benefit of a distant area, separate and apart from the area in which his land was located, would he then be entitled to recompense for what might be called, for want of a better term, involuntary condemnation?

Assume two ground water basins, one of which receives a portion of its annual recharge by underflow from the second, or higher, basin. The lower basin has a historical safe annual yield of 40,000 acre-ft per yr. This is insufficient to provide for the requirement of the overlying lands and the landowners have consequently been considering the purchase of a supplementary supply of 10,000 acre-feet per yr at a cost of $10.00 per acre-ft. The upper basin forms a ground water recharge district and secures a supply of water for the purpose of recharging the underground aquifers. As a result of this recharge, the underflow to the lower basin is increased by 10,000 acre-ft per yr. This increase relieves that basin of the necessity for an annual payment of approximately $100,000 for this quantity of water.

Because it is not possible to prevent the movement of water underground in the direction of the hydraulic gradient, the courts in some jurisdictions, notably in California, have followed the rule of correlative rights in percolating waters and have attempted to identify all users drawing from a common supply. The question that arises in cases similar to that previously described is, how should the extent of the boundaries of ground water recharge districts be determined? Should they be limited to single basins or should they be extended to cover all possible areas which are, or may be, affected by underflow from any part of the entire area?

Another question that arises in the planned operation of groundwater basins is the case of a basin in which a portion of the surface area, lying at lower elevations, was, in a state of nature, swamp land. The swamp area resulted from rising water in the lowermost portions of the basin. After commencing pumping operations, the water table beneath the swamp area was lowered to a considerable extent, the land was reclaimed, settled, and large investments were made in farming enterprises. A ground water recharge district is now organized and, depending upon the boundary chosen, it may or may not include this former swamp area. Water is imported from sources outside the basin and percolated to the underground aquifers for storage. The water level rises, and although the lands in the upper part of the basin are still in excellent condition for farming operations, those in the lower part become subject to high water table conditions and even, in some instances, to standing water. Because this is a natural phenomenon, do the owners of these lands have any basis for recompense? Would they be entitled to the maintenance of a condition conducive to intense utilization, resulting from the original pumping of water for use on higher lands and on which they relied when developing their own lands?

An important legal problem associated with recharge operations is the identification of the water that has been placed in underground storage. Another is the determination of the effect of recharge activities on the basin and on the water users deriving their supply from the underground sources. It is necessary to know how much water has percolated, and its direction and rate of travel. It is also necessary to ascertain the effect of recharge operations on the basin so that determination of injury or benefit, if any, to overlying owners can be made. Water quality aspects of the recharge operation are another extremely important factor. Because of the deficiencies in the present state of knowledge, the variables involved, and the scope of the required data collection and research activities, definitive conclusions on the effect of groundwater recharge activities are not easily determined.

The deliberate artificial recharge of a ground water basin is recognized in the California Water Code, which states;

> "The storing of water underground, including the diversion of streams and the flowing of water on lands necessary to the accomplishment of such storage, constitutes a beneficial use of water if the water so stored is thereafter applied to the beneficial purposes for which the appropriation for storage was made." (Section 1242)

The principles underlying the statutory authority have been recognized and upheld by the courts in the case of City of Los Angeles versus City of Glendale, 142 P. (2d) 289. The City of Los Angeles has, for many years, imported water into southern California from the Owens River basin, to the east of the Sierra Nevada. A considerable quantity of the imported supply percolates to the underground basin as a result of both deliberate and incidental artificial recharge operations. The cities of Burbank and Glendale established well fields drawing from the underground supply. Los Angeles then sued to prevent the other municipalities acquiring a right to the imported supply conserved in such underground storage. In its decision, the court held,

> "(The City of Los Angeles) had a prior right to the use of water brought to the San Fernando Valley. It did not abandon that right when it spread the water for economical transportation and storage . . . once within the

> basin, enroute to diversion works, it was in effect within (the city's reservoir , . . It would be harsh to compel (the city) to build reservoirs when natural ones were available . . . Part of (the water) was spread . . . in gravel pits and "spreading grounds" . . . the remainder . . . was sold to the farmers of the San Fernando Valley, and as forseen . . . $27\frac{1}{2}$ per cent of it also sank beneath the surface and joined the normal and spread waters."

Other cases bearing on the right of recapture of seepage or return water after it has left the boundaries of a project (providing such recapture can be proved to have been a part of the planned operation of a project) are Ide versus United States, 263, U. S. 497, involving a water project in Wyoming; and an Idaho case, United States versus Haga, 276 Fed. 41.

Organization for Ground Water Basin Operation.—It is generally agreed that all who benefit from a project, such as the recharge of the ground water basin, should share equitably in the costs. Existing irrigation, water conservation, and similar districts were generally organized to make surface water supplies available to the district area. Consequently, the boundaries of such districts frequently do not coincide with the boundaries of underground water basins. Districts covering a portion of a ground water basin frequently find that deliberate or incidental artificial recharge, resulting from their activities, produces appreciable benefits for many landowners outside the district by reason of the rise in the ground water level.

An exact solution for this difficulty would be the organization of a district with boundaries corresponding with the limits of the underground basin. Where no existing districts are organized, this would be a simple process. In areas where part of the basin is included in organized districts, an overall district might be established, either replacing or including existing agencies. Where inclusion is effected, however, difficulties due to dual taxation of lands in the existing districts might ensue. A possible remedy might lie in the establishment of a new district in the unorganized area only, followed by organization of a supervising, or coordinating, district having only a small staff and no operating responsibility.

Appropriate district organizations for ground water development will depend, of course, on the legislative authorizations incorporated into the law of various states. Irrigation and other types of districts often have general powers that may be interpreted to include the recharge of ground water basins and the performance of the conjunctive operation. Such districts, however, may encounter problems in the distribution of costs in accordance with benefits in cases where the benefits are unequally distributed. This is particularly important where benefits transcend district boundaries. Although a rise in the water table would tend, in some instances, to affect land values, the result would not necessarily be proportional to location or size of parcel.

Legislation authorizing the formation of ground water operating districts should provide for the following: (a) the method of forming the district; (b) the appointment or election of a governing board; (c) the purchase of, or other adjustment of conflicts with, existing water rights; (d) the conservation of water supplies by maximum use of the available storage, including artificial recharge; (e) the development and transportation, or purchase, of water supplies from without the basin; (f) the development and distribution of water supplies within the basin; (g) the imposition of general taxation in order to share the burden of costly supplies among those who ordinarily secure their supplies by other means; (h) the treatment and conservation of waste waters, (i) the coordinated and conjunctive operation of ground water with surface storage; (j) special arrangements with, or assessments on, or both with those continuing to pump from the underground basin; and (k) sound methods of financing for needed works and improvements.

In California, legislative authority and sanction has been given to a form of district organization specifically modeled to meet the peculiar requirements associated with ground water basin operations. These districts, known as Water Replenishment Districts, may be organized in regions where the existing underground water supplies are insufficient to meet demands placed upon them and where further excessive pumping without replenishment will destroy the usefulness of the underground reservoir. At the present time, the organization of such districts is limited to the arid southern part of the state, although it is anticipated that the legislation will be made applicable to other areas as the need arises.

Formation of water replenishment districts may be commenced by petition signed by at least 10% of the registered voters residing in the proposed district. The completed petition is referred to the State Department of Water Resources for action. The Department makes an appropriate investigation, followed by hearings, for the purpose of determining (a) whether the persons or lands in the proposed district will actually benefit by planned utilization of the ground water basins, and (b) whether the boundary of the proposed district includes all persons and lands that rely on the underlying ground water supplies. The Department may modify the boundaries of the proposed district to include or exclude certain lands in accordance with the benefits derived. An unfavorable finding will bring the proceedings to a half, although a new petition may be filed after a lapse of at least 6 months. A favorable report by the Department, however, will set in motion the statutory proceedings required to complete the organization of a district.

Water replenishment districts, as authorized in California, may levy assessments in proportion to the quantity of water pumped from the underground reservoir. This is important in providing for the equitable assessment of the benefit to holders of appropriative and prescriptive rights to use water on non-overlying lands. The districts also have the power to purchase water from sources outside the district and distribute such water in exchange for the cessation or reduction of ground water extractions. This permits replenishing and managing the ground water supply within the district. The districts may also spread or inject water into underground aquifers and "store, transport, recapture, reclaim, purify, treat, or otherwise manage and control water for beneficial use of persons or property within the district."

Administration and Control of Ground Water Utilization.—In all states in which planned river basin development, including ground water utilization, will take place, some form of water rights doctrine already exists. This doctrine may be solely the appropriation doctrine for surface streams, or it may be a combination of appropriation and riparian doctrines, as in California. In the eastern United States, the riparian doctrine generally prevails for surface water supplies, whereas ground water is usually governed by either the "absolute ownership" or the "reasonable use" doctrine.

This being the case, it is imperative that legislation promoting ground water utilization provide adequate protection to prior rights to the use of water and establish an equitable basis for compensating owners of rights that are acquired in order to accomplish successful operation.

The recognition and ratification of existing rights is generally accomplished by "saving" clauses in pertinent legislation. The question of compensation, however, may be handled on an individual basis by the courts, that may lead to inequitable settlements with different parties in cases where the facts are essentially similar, or may be guided by principles laid down in the law and administered by either a single agency or by the courts. The procedures generally followed in the valuation of lands and improvements acquired for public purposes would seem to have applicability insofar as acquisition of such classes of property would be required. The valuation of a water right held by an overlying owner, however, would appear to be somewhat more difficult than appraising an item of tangible property.

A suggested method of evaluating this right, consists of establishing the historical annual cost of pumping, per acre-foot, to the overlying owner and subtracting such cost from the proposed price at which a supply will be furnished to his lands. The balance would represent the additional cost of such replacement supply. This amount could then be capitalized for an appropriate period, 50 yr, and the resulting total termed the value of his right. This, then, would be the amount he would receive as compensation.

Legislation providing for the increased use of groundwater storage capacity in the conservation and regulation of water supplies should also provide for the means of properly supervising such utilization; for the organization of operating agencies (as previously examined); for financing such operations on a sound and logical basis; and for the control of the quality of water in underground storage.

It is felt that, because of the oftimes large geographical coverage required for ground water operations and the frequent necessity of coordinating conservation activities in widely separated regions, contol and supervision should not be delegated to an agency of less than statewide authority. It is not necessary that such agency actually design, construct, or operate conservation projects, although that might be desirable in certain circumstances. The agency should, however, have the responsibility to (a) review, (b) approve, modify, or disapprove, (c) supervise, (d) coordinate, and (e) regulate the organization, plans, financing, construction, and operation of districts or other agencies engaged in the development and utilization of ground water for beneficial use. This would ensure uniform standards for the protection of the public interest in the development of vitally needed water supplies.

The same, or at least a second, state agency should have the responsibility for protecting the quality of ground water supplies. This should extent to regulatory authority over the disposal of liquid or other wastes that would tend to adversely affect water quality; and authority to restrict, or otherwise control, ground water operating practices that would permit the intrusion of degrading waters into otherwise usable supplies. If this latter agency is not a part of the overall control authority, explicit legislative provisions for adequate coordination between the two agencies would be required.

A thorough review and analysis of the engineering, financial, and economic feasibility and justification of proposed ground water recharge and conservation projects will do much to enhance their desirability and attractiveness to the public. Many of the legal considerations involved in ground water development will necessarily be solved in the project formulation process. Comprehensive basic legislative authority, however, will be the cornerstone on which the maximum conservation and development of water resources, particularly ground water resources, will be accomplished for the greatest benefit to all concerned.

ACKNOWLEDGMENTS

The opinions and concepts expressed herein are those of the writer and are not to be construed to represent the policies or programs of the California Department of Water Resources. Grateful acknowledgment is made to the many who have, wittingly or unwittingly, contributed to the material contained herein. Special appreciation is due Martin McDonough, Attorney-at-Law, Sacramento, California, and to Russell R. Kletzing, Associate Attorney, California Department of Water Resources, who reviewed and commented on the original manuscript.

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DISCUSSION

RAPHAEL G. KAZMANN,[4] **F. ASCE.**—Thomas has done an outstanding job in bringing together a discussion of legal principles in view of hydrogeologic fact. The author's criticisms of the University of Michigan's "Model Law" are well founded and the writer would like to enlarge upon them.

"(2) Existing riparian uses are confirmed, but those which have never been exercised are disallowed."

The principal difficulty with riparian doctrine is that no one can count on using water. By "using" is meant either to evaporate it or to degrade it in quality—as by heating it, or increasing its mineralization or organic content. The sanctioned riparian "uses" have less effect on the flow of a stream than do the natural variations of evaporation and rainfall. Modern industrial and agricultural practice, however, alters or consumes water. It seems to the writer that the whole basis of operation of the economy, including its expansion, requires a sound legal framework for the acquisition of a property right in the use of water. This property right could be sold, transferred, or condemned in the same manner as any other property right. Appropriation doctrine is the only tested basis for water resources investments because it gives assurance that water will be available for beneficial use insofar as nature permits, without consideration of the changing definition of "beneficial" put forward by some bureaucratic authority. Long range investments in water-using facilities are, therefore, justified. And this brings us to:

"(3) Except for existing uses of water, which are unlimited in duration, fifty years is the maximum terms for which a right of water use may be secured."

In the writer's opinion there is nothing more calculated to promote confusion and uncertainty as regards the allocation and utilization of water resources than this arbitrary restriction, now embodied in the "Model Law," that actually seems to constitute a new sort of property right. We can readily visualize the periodic reopening of old rivalries and the constant off-season political struggle, as the control of water resources periodically "comes up for grabs" by public organizations, individuals, governmental bureaus, and corporations. This built-in reopener clause is a pernicious provision which will not serve to develop the beneficial use of water but will serve to promote turmoil. A water right should be in the same category as ownership of land or other property.

These comments, of course, reinforce the author's criticisms (4), (5), (6), and (7). As Thomas points out, the omission of a provision for the conjunctive use of ground and surface water supplies is a serious lack.

In general, it has always seemed strange to the writer that the University of Michigan, located in a state lacking in the experience of water law, should feel called on to draft a "Model Law"—not just a law applicable to Michigan—whereas law schools located in states having from 20 yr to 50 yr of experience in the operation of water legislation are far more hesitant in this regard.

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[4] Cons. Engr., Stuttgart, Ark.

An intensive study of the publication issued by the University of Michigan Law School, entitled, "Water Resources and the Law," indicates to the writer that the objective of their "model water law" is to give to an administrator complete power over the use of water by anyone, organization or person, on the basis of "beneficial" use, with the administrator defining what he means by "beneficial" at any particular time. Thus, the "Model Act" discards predetermined external standards. This proposed grant of arbitrary power is both undesirable and dangerous.

All existing water-use legislation can be characterized as being "water oriented." This means that the underlying assumption is made that the beneficial utilization of the water substance itself is the only matter to be considered. By no means should this assumption go unchallenged. Actually, in many instances the most beneficial use of water is to get rid of it. This is true to some extent in the production of every kind of mineral—the dewatering of ore bodies is an essential part of the mining process. It is well to remember that our modern civilization rests upon an ever-broader base of mineral utilization and upon the commercial development of ever-leaner rocks and ores. This means that the disposal of water from the workings can be expected to be a problem of increasing importance.

To the knowledge of the writer, the "beneficial" waste of water has not been provided for in any water-use legislation. The "corpus" of the water, under existing legislation, must in itself be utilized.

Under the doctrine of appropriation, there is always the possibility that the state engineer, or other proper authority, may be able to declare that the disposal of water from mines or excavations is a beneficial "use" of the water. Even so, it is not at all apparent on what basis the seasonal variations in ground-water flow, or the increases in ground-water flow due to extension of the mine workings, are to be handled.

In states where the riparian doctrine, or some possible modification of it, is more or less in force, the status of ground-water developments, as well as mine dewatering, is unclear. The "model" law never comes to grip with the issue of the beneficial waste of water.

In essence, therefore, the writer subscribes to the tenor of Thomas' remarks on the legal aspects of ground-water utilization. It is the writer's opinion that the "model" law, as it is now proposed, is a pernicious piece of legal draftsmanship that is totally unprincipled in that it gives to unknown officials the absolute and arbitrary right to declare who shall use water and when. The officials can do this in the name of the "public good" by declaring that one use of water is "beneficial" as compared to another.

The real problem is to extend the operation of our present system of property rights to the field of water utilization and, thus, bring water more efficiently into the process of production, not to grant arbitrary powers of life and death to faceless administrators in the name of the public welfare. Thomas has done the profession and the country a service by expressing the real problems in the field of water development clearly and adequately.

FREDERICK L. HOTES,[5] F. ASCE.—An excellent summary of a voluminous topic has been presented by Thomas. Most important, he has focused attention on some of the significant problems that today face the engineer who plays a prominent role in the conservation and development of the ground waters of the United States. These problems, however, are not within the exclusive province of the engineer. They belong also to the geologist, the lawyer, the jurist, the public official, the economist, and above all, to the man on the land who uses these waters to gain his livelihood.

Recognizing the broad cross section of life that is concerned with ground water problems, the writer wonders if Thomas is not being unduly harsh with the legal profession when he implies that many of their decisions "have often served only to complicate the use and development of ground water." Certainly the basic intent of any judicial decision is to clarify, not to confuse, although it is recognized that this objective is not always obtained. The fact that legal provisions and decisions are often complicated is due perhaps more to the inherent complexity of ground water hydrology, rather than to any other cause. Often the jurist must resolve conflicting expert testimony on technical matters; because those who should be best qualified to determine and interpret the facts, cannot themselves agree. There will never be a ground water code that can adequately cover all possible circumstances. Interpretations and judgment, on the part of engineers as well as others, will always be required. Improvements in water law and regulations, however, will continue to accrue, as in the past, from the joint efforts of engineers, scientists, lawyers, and laymen, to enlarge knowledge and understanding of both technical and nontechnical aspects of the field.

Complex water law is not solely the affliction of the highly-developed civilization of the western United States. The writer has had experience in the Middle East with equally complex customs, traditions, and laws governing the use of surface and ground waters. Irrigation has been a vital portion of the life of this region since long before the time of the Roman Empire. Here, water is used in accordance with tribal, religious and governmental law, written and unwritten, developed over centuries. Whereas details of the law differ even more widely over that area than within the United States, and practically none of it is codified, the basic elements of our riparian and appropriation doctrines can be readily identified everywhere. Wherever the irrigation engineer works he finds a knowledge of local water law necessary for the formulation of practical development plans. Hence, the appropriateness of papers such as the one under discussion, to appear at times in the technical journals of the Society.

In his analysis of the Model Water Use Act, Thomas, under item (4), mentions a possible priority list of beneficial uses. The writer agrees that this should not be an administrative prerogative, but believes that for the most important uses it is a necessary legislative prerogative and duty. California law clearly establishes two relative priorities; domestic being the highest use, and irrigation being the next highest use. Nothing is stated concerning industrial use, one of the significant beneficial uses. There are indications that the term "municipal use" has been considered as a type of domestic use, even though industrial water demand may be as much as one-third of the total municipal demand. With economic competition for water increasing, the question arises as to the relative priority of industrial and irrigation uses.

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If an industry were to appropriate water in its own name, its priority of use, in case of water shortage, or for the granting of a new right, would be less than that of an irrigation use. If, on the other hand, a municipality were to secure the right to the water, and in turn supply it to the industry, the same physical use of the water would have precedence over irrigation use, thus reversing the apparently stated priority of irrigation over industrial water use. To the writer's knowledge, this issue has never yet been brought before a court of law. Is it proper to interpret the foregoing to mean that the industrial use of water within a municipality is as vital to the life of the city as its domestic use, but that industrial use not derived from a municipal supply is not as important? Legislative definition and clarification may be required ultimately.

Thomas raises the question as to what ground water appropriators in New Mexico can do after exhausting their water supply. There are three alternatives open to them, namely,

1. Import supplemental water. This may be uneconomic.
2. Reduce draft to safe yield. This will automatically occur once the basin is depleted.
3. Sell-out. Let someone else worry about the application of alternatives one and two.

The author has ended his article with the strong implication that ground water operations will be subjected more and more to administrative and executive, as well as judicial, control by State governments. There are alternatives, such as local governmental units and water user associations, but these may not be able to provide the legal, financial, and technical effort required to match the problems of the future. Even more important, they may not be willing to do so, although theoretically able to do so. Such being the case, the Progress of the Commonwealth will generally proceed by the best available remaining course. Where the State Government also is unwilling or unable to act, then the Federal Government may be the next logical alternative. Examples of all types exist today. It will be interesting to observe the trend of the immediate future.

PAUL BAUMANN,[6] F. ASCE.—Thomas is to be complimented on his review of water right evolution, in general, extending over the past century, and the more recent legal aspects of ground water utilization, in particular.

Successful enactment, in 1958, by the California Legislature, of statutes authorizing the formation of (ground water) Replenishment Districts should, above all, be credited to the foresight, initiative, and leadership of the West and Central Basins Water Associations, in Los Angeles County. These associations, and particularly the former, were, likewise, instrumental in the enactment, in 1951, of an amendment to the Los Angeles County Flood Control District Act, (originally created by the California Legislature in June, 1916) which authorized said District to establish water conservation Zones.

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Subsequently, two of such Zones were established, Zone I, generally overlying the Central Coastal ground water basin, and Zone II, the boundaries of which generally coincide with the boundaries of the West Coast ground water basin. In brief the purpose of these Zones was to produce a revenue through taxation, limited to five cents per hundred dollars assessed valuation with which the Flood Control District could purchase water from outside sources to be conserved through spreading or other means within the revenue producing Zone.

Prior to this amendment the act provided for the conservation, but not the purchase, of waters by the Flood Control District. Hence, the Zones were a forerunner of the Replenishment Districts, one of which comprising both West Coast and Central Coastal Basins, was formed in 1959.

Through a fine distinction in the wording of the Zone versus the Replenishment District provisions the respective boundaries could be at variance. Within the boundary of the Zone all land was to be included which was specially benefitted through extractions of water from the ground water basin, whereas within the boundary of the Replenishment District all land thereby benefitted was to be included. The legal interpretation of "special benefit" was the predominant (more than 50%) use of extracted ground water.

In a constructive manner, Thomas sets forth legislation necessary to supplement current provisions or practices in ground water utilization in light of available basin storage and recharge. To the writer's knowledge, little difficulty has so far been encountered on the part of owners of land overlying a ground water basin in putting water extracted therefrom to use on his land even if such use could not be considered strictly beneficial. However, the same owner of land is likely to take exception if his neighbor should extract ground water for use not on overlying land even if such use happened to be strictly beneficial, such as domestic use. Recent litigation in the southern part of the state of California was a case in point.

A water company with large holdings overlying a ground water basin had been extracting and exporting ground water for domestic use. Farmers overlying the same basin had been extracting ground water for irrigation and other agricultural use on their land. All was well until the current drought started, in around 1943, and with it the depletion of the ground water supply began, which depended principally on stream flow percolation besides rainfall on the area itself.

Like many southwestern streams, the one in question would produce occasional flash floods and little if any flow in between. Although such flash floods became fewer and farther apart as the drought progressed, a considerable volume of potable water would waste to the ocean due to lack of absorption capacity of the streambed. Hence, the depletion of the basin continued and the case was taken to court. Here a number of ideas were propounded in regard to ground water rights which in light of the author's outlines and suggestions are believed to be worthy of mention. They bring out the fact that ground water rights are still vague and indefinite.

The water company offered to construct spreading grounds capable of conserving most of the stream flow which would otherwise waste to the ocean and to substantially limit its exporatation of ground water to the quantity so conserved. The spreading grounds would be located on company property.

The opposition countered that although spreading were performed on company property, the water so conserved would not necessarily be stored exactly underneath said property and that the company had no right to store it anywhere else. Furthermore, in extracting water from the basin the company had no right to extract any water, except particle by particle the very same water it had conserved.

While on the face of it these arguments sound far fetched if not harebrained, they nevertheless cannot be denied the claim of being food for thought.

ROBERT O. THOMAS,[7] F. ASCE.—The discussion of the basic paper has served to illustrate and emphasize the multi-faceted nature of the problems encountered in the development and utilization of ground water. Although the discussions were few in number, each of the discussers has been prominently identified with ground water development in the course of his career and their remarks carry the weight of authoritative experience. It is unfortunate that no discussion was received from a member of the legal profession, although many copies of the paper were circulated to that end.

Kazmann raises the question of the right to dispose of unwanted or undesired water. While there has not been, so far as the writer is aware, any legislation providing for the acquisition of the right to the "beneficial waste" of water, the fact of such waste has been recognized in legislation directed toward other purposes. Typical instances where this has occurred are laws relating to the construction and operation of flood control facilities; the recognized right to protect oneself against the elements; laws providing for the drainage and reclamation of swamp and overflow lands; and water pollution control legislation.

The latter recognizes, by implication, that liquid waste, such as the frequency acid mine drainage and the brines produced in oil development do, in fact, exists. Pollution control legislation provides the means of regulating such discharges to alleviate damage to otherwise usable receiving waters. It appears to the writer that the required wastage of certain portions of our water supplies has heretofore been principally provided for under governmental police and welfare powers and functions and by civil actions for damage, when such result has been associated with the means of disposal. It may be, however, that the increasing complexity of our civilization may eventually require specific legislation relating to what is termed, by Kazmann, "beneficial waste."

Hotes very properly points out that ground water problems are not the exclusive province of the engineer. The writer has recognized this fact repeatedly[8] and can only plead that this paper was limited to a single aspect of a wide-ranging subject. The writer agrees that much of the apparent legal confusion with regard to ground water is traceable to the disparity of opinion generally presented for the consideration of the court. This could, of course, be minimized if the oft-repeated suggestion that factual physical and technical testimony should be presented by qualified scientists and engineers appointed directly by the court, with fees assessed equally to each of the litigants, is followed in practise.

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[7] Water Resources Engr., U.S. Tech. Cooperation Mission to India, New Delhi, India; formerly, Superv. Hydr. Engr., State Dept. of Water Resources, Sacramento, Calif.

[8] Discussion by Robert O. Thomas, of "Ground Water Problems in New York and New England," Proceedings, ASCE, Vol. 85, No. HY 11, November, 1959.

Hotes has quite properly raised the question of the priority of industrial use of water. During the distant, and recent, past, when our theories of water rights were in the process of formulation and codification, the principal municipal use of water was for domestic purposes. Most of the industrial use which occurred prior to the period of the Civil War consisted of using water supplies for the purpose of producing power, such as in the operation of water wheels. With the growth of mass industry, the transfer of food product processing from the home to the factory, and the advent of new chemical products and processes, the present-day use of water by industry is frequently a major portion of the municipal requirement. With ever-increasing development of our water resources, the question of priority of water use between the industry which gives employment and the farm which produces food will, in many cases, become of prime importance.

The writer is appreciative of Baumann's contribution, particularly with regard to the proper assessment of credit for the origination and support of water replenishment district legislation in California. The original paper, of course, was general and impersonal in nature, and Baumann's review constitutes a valuable addition to the history of water legislation in this country.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3242

MEAN RESIDENCE TIME OF LIQUID IN TRICKLING FILTER

By Morton D. Sinkoff,[1] A. M. ASCE, Ralph Porges,[2] F. ASCE,
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SYNOPSIS

Pilot plant research studies aimed at broadening the basic knowledge relative to the performance of trickling filters were initiated. Herein are presented the results of initial hydraulic studies in which the mean residence time was determined for tap water applied to columns packed with spherical media under conditions of varying hydraulic loading rates, media size and depth.

BACKGROUND

The objective of the trickling filter research studies at the Robert A. Taft Sanitary Engineering Center is to broaden basic knowledge relative to the performance of trickling filters through analysis of experimental data secured from carefully designed and controlled experiments. It is believed that this knowledge will lead toward better designed, more effective, and more economical trickling filters.

For over half a century trickling filters have been utilized for treatment of wastes that are amenable to biochemical stabilization, consequently, the literature contains vast numbers of operational reports and theoretical discussions relative to the design, performance, and evaluation of trickling filters. Most

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Note.—Published essentially as printed here, in November, 1959, in the Journal of the Sanitary Engineering Division, as Proceedings Paper 2251. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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of the past studies were performed on full scale filters in which individual variables could not be controlled and frequently were not considered or were not susceptible to measurement. Small laboratory studies at colleges and universities, as a result of limited amounts of time, money, and personnel, were generally restricted in scope to determination of the individual influence of one variable upon filter performance. However, although considerable time and effort has been expended by past workers, many questions are still unanswered and there is no commonly accepted theory of performance or method of performance evaluation based on both practical and theoretical grounds.

The United States Public Health Service (USPHS) considered a comprehensive pilot plant investigation necessary in order to study the cumulative influences of the many variables affecting the trickling filter process. The study reported herein is the initial step in this basic research program.

INTRODUCTION

It seems reasonable to theorize that the degree of purification obtained in a trickling filter is, in some manner, proportional to the length of time provided for biochemical change within the bed. This is by no means a new concept and much postulation and research along these lines have been done (5-16).[4] The initial research project was designed to determine if a correlation exists between the mean time of passage of waste liquor through a trickling filter (mean residence time) and the degree of purification attained.

A comprehensive trickling filter study aimed at developing design criteria should include investigation of the effect of the following variables on filter performance: Hydraulic and BOD loading; amenability of feed to treatment; temperature of feed; media characteristics, including depth, size, shape, roughness, porosity, and voids; slime types and volumes of zoogloeal masses; type of distribution, distribution frequency, and recirculation. These factors were divided into three general classifications: The physical system, the biological system, and the biophysical system. The study of the physical system is directed to the determination of the effects of the physical dimensions of the system on its hydraulic characteristics; the study of the biological system will be directed primarily to the phenomena of purification; and the research on the biophysical system will endeavor to correlate the combined effects and interaction of the physical and biological parameters. Due to the complexity of the problem, it was thought advisable to perform separate research on each of the three phases mentioned. The study reported on herein, Phase I, is the initial study of the physical system dealing with the hydraulic characteristics of flow through a bed of granular media aimed at determining the mean residence times of the liquid portion of a waste (simulated by using tap water) through a clean bed of media.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference, in Appendix II.

Mean Residence Time.—The time that a particle of fluid remains within a filter column is defined as the residence time. When a liquid is applied to the column, as in the application of sewage to a trickling filter, various portions of the liquid follow different flow patterns. With lateral and longitudinal mixing, different residence times result for different particles of fluid even though they were part of the same mass of fluid when applied to the column. The mean residence time is defined as the average residence time for all individual particles applied to the bed during a finite interval of time.

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[4] Numerals in parenthesis refer to corresponding items in the Bibliography. See Appendix III.

The mean residence time may be measured as follows: Let a finite quantity of tracer material be applied instantaneously at time $t_o = 0$, to the stream of fluid applied to the top of the bed. At some chosen depth, whether it be some intermediate one or at exit, the fluid is sampled and the concentration of tracer in each sample is determined. A curve of tracer concentration versus the elapsed time from t_o is plotted. Assuming that the tracer acts identically as the fluid, then the mean residence time (t_r) is, by definition, the displacement of the centroid of the area under the concentration (c) versus time (t) curve from the c axis (along the time axis) see Fig. 1. Expressing this concept mathematically

TABLE 1.—VARIABLES IN EXPERIMENTAL DESIGN

Variable (1)	Symbol (2)	Units[a] (3)
1. Rate of fluid application	Q_1	L^3/T
2. Viscosity of fluid	μ	$\frac{M}{LT}$
3. Density of fluid	ρ	$\frac{M}{L^3}$
4. Surface area of bed	A	L^2
5. Depth of bed	H	L
6. Size of media	S_1	L
7. Packing density of media	S_2	——
8. Acceleration due to gravity	g	L/T^2
9. Roughness of media	R	——
10. Shape of media	S_3	——
11. Surface tension of fluid	σ	M/T^2

[a] M = mass, L = length, and T = time.

$$t_r = \frac{\int_0^\infty c\ t\ dt}{\int_0^\infty c\ dt} \qquad \qquad (1)$$

The area under the curve, $\int_0^\infty c\ dt$, is proportional to the quantity of tracer applied. That is, $Q\ A \int_0^\infty c\ dt$ equals the amount of tracer appearing in the effluent, in which Q is the hydraulic loading rate and A denotes the bed. The

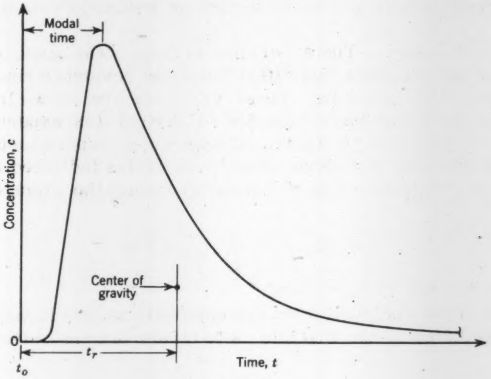

FIG. 1.—CONCENTRATION—TIME CURVE

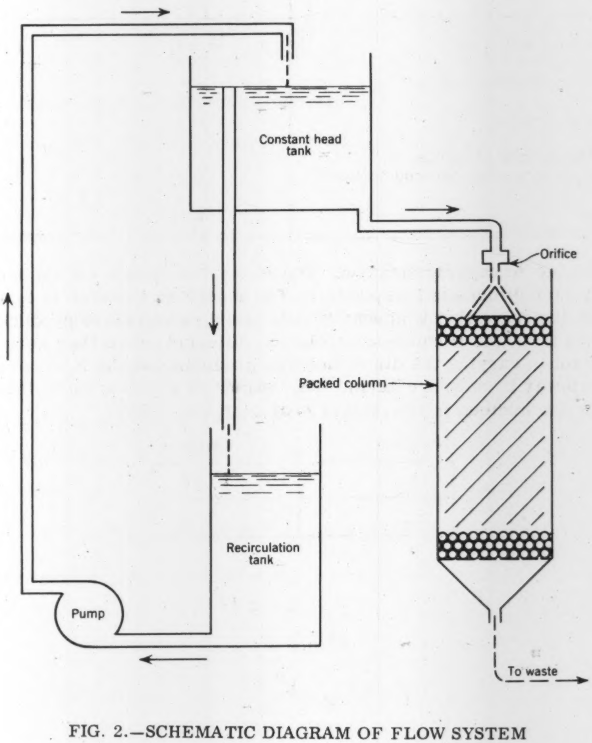
Constant head tank
Orifice
Packed column
Recirculation tank
Pump
To waste

FIG. 2.—SCHEMATIC DIAGRAM OF FLOW SYSTEM

amount referred to here can be by weight or volume, depending on the units chosen.

Experimental Design.—The experimental design was based on a dimensional analysis (1) of the variables that effect the mean residence time (t_r) of a fluid in a packed bed of clean media. These variables are shown in Table 1. Only the first eight variables were included as part of this experiment: The remaining three items will be studied in subsequent experiments. Refining the notation, t_r is assumed to depend upon the variables indicated in Table 2.

Dimensional Analysis.—The relationship among the variables is indicated by

$$f_1(Q, g, \nu, H, S, t_r) = 0 \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which f ( ) denotes "a function of."

The choice of the nucleus for the dimensional analysis is important because one of the objectives of the analysis is to obtain dimensionless products that are amenable to experimentation. The variables chosen as the nucleus will appear in each dimensionless product. The dependent variable is t_r and, therefore, it is desirable that it appear in only one dimensionless product. Because the factors Q, S, and H can be controlled at different levels they serve as quantities useful in varying the dimensionless products making it advantageous for each to appear in only one term. The quantities g and ν were therefore selected for the nucleus in the dimensional analysis. Thus;

TABLE 2

Variable (1)	Symbol (2)	Units (3)
1. Hydraulic loading rate	$\frac{Q_1}{A} = Q$	L/T
2. Kinematic viscosity of fluid	$\mu/\rho = \nu$	L^2/T
3. Depth of column	H	L
4. Specific surface of media, Media surface area/Volume occupied	S	1/L
5. Gravity	g	L/T^2

Nucleus	Units
g	L/T^2
ν	L^2/T

then

$$L = g\,T^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3a)$$

and

$$T = L^2/\nu = \frac{g^2\,T^4}{\nu} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3b)$$

$$T^3 = \frac{\nu}{g^2} \tag{4a}$$

$$T = \nu^{1/3}\, g^{-2/3} \tag{4b}$$

and

$$L = g^{-1/3}\, \nu^{2/3} \tag{4c}$$

Then, using the notation after E. Buckingham (2),

$$\pi_1 = \frac{t_r}{\nu^{1/3}\, g^{-2/3}} \tag{5a}$$

$$\pi_2 = \frac{Q}{g^{1/3}\, \nu^{1/3}} \tag{5b}$$

$$\pi_3 = S g^{-1/3} \nu^{2/3} \tag{5c}$$

and

$$\pi_4 = \frac{H}{g^{-1/3}\, \nu^{2/3}} \tag{5d}$$

and according to the Buckingham pi theorem

$$\pi_1 = f_2\,(\pi_2, \pi_3, \pi_4) \tag{6}$$

or

$$\frac{t_r\, g^{2/3}}{\nu^{1/3}} = f_2\left(\frac{Q}{g^{1/3}\, \nu^{1/3}}, \frac{S\, \nu^{2/3}}{g^{1/3}}, \frac{H\, g^{1/3}}{\nu^{2/3}}\right) \tag{7}$$

Assuming that t is directly proportional to H we may write

$$\frac{t_r\, g^{2/3}}{\nu^{1/3}} = \frac{H\, g^{1/3}}{\nu^{2/3}}\, f_3\left[\frac{Q}{g^{1/3}\, \nu^{1/3}}, \frac{S\, \nu^{2/3}}{g^{1/3}}\right] \tag{8}$$

or

$$t_r = \frac{H}{g^{1/3}\, \nu^{1/3}}\, f\left[\frac{Q}{g^{1/3}\, \nu^{1/3}}, \frac{S\, \nu^{2/3}}{g^{1/3}}\right] \tag{9}$$

For brevity, reverting to the π notation,

$$\frac{Q}{g^{1/3}\, \nu^{1/3}} = \pi_2 \tag{10a}$$

$$\frac{t_r\, g^{1/3}\, \nu^{1/3}}{H} = \frac{\pi_1}{\pi_4} \quad \ldots\ldots\ldots\ldots\ldots \quad (10b)$$

and

$$\frac{S\, \nu^{2/3}}{g^{1/3}} = \pi_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10c)$$

Eq. 8 becomes

$$\frac{\pi_1}{\pi_4} = f\,(\pi_2, \pi_3) \quad \ldots\ldots\ldots\ldots\ldots \quad (11)$$

On the basis of this theoretical design, the results will plot as a family of curves displaying Eq. 11 when $\frac{\pi_1}{\pi_4}$ is plotted against π_2 for different values of π_3.

PHYSICAL DESIGN

General.—In order to evaluate the functional relationship set up by dimensional analysis (Eq. 11), π_2 was varied against $\frac{\pi_1}{\pi_4}$ with π_3 constant and π_3 was varied against $\frac{\pi_1}{\pi_4}$ with π_2 constant. This was accomplished by performing a set of experiments on a column of known media size while varying the hydraulic loading rate and measuring t_r. The latter criteria may then be accomplished by performing the same experiment a number of times on beds of different size media. In essence, then, our independent variables will be Q and S, with t_r the dependent variable. Therefore, as wide a range of Q and S as practical was desired. Also, it was necessary to keep the shape of the media constant with different sizes so that an evaluation of shape would not be necessary during this experiment. Similarly, media roughness variations were held to a minimum because this factor was neglected in the dimensional design.

Media.—To meet the design criteria for the media, smooth spheres were chosen. They were 1/2-in., 3/4-in., and 1-in. diameter clean, smooth, glass spheres and 3-in. diameter clean, smooth, non-porous porcelain spheres. Each size packed in a different column. The choice of 3-in. porcelain spheres rather than glass spheres was a practical matter as the cost of 3-in. glass spheres was prohibitive. A different diameter column, proportional to the sphere size, was chosen for the purpose of minimizing wall effect. Table 3 presents the physical characteristics of the columns and media pertinent to the experiment.

Application Rates.—The choice of flow rates depended on physical limitations and the desire to investigate flow rates in the range 1-to-200 million gallons per acre per day (MGAD). To facilitate metering flow over a wide range, a constant head tank was installed to supply flow to each column, the flow being varied by use of calibrated orifices. It was assumed that t_r would be a power function of Q, and, therefore, a geometric spacing of the rate of application was established.

Tracer.—To ascertain the mean residence time of the passage of fluid in the column, a suitable tracer was needed. The basic criterion in the selection of the tracer was that it should exhibit the same hydraulic characteristics as the fluid.

Three tracers were considered - radioactive, organic dyes, and inorganic salts. Radiotracers were ruled out because of cost. Dyes, fluorecein and Congo Red, were investigated experimentally. It was found that the dyes adhered to the media causing significant drag-out. Sodium chloride was selected among the salts considered for use in the study when it was found that if fulfilled the criteria. Salt density currents cannot occur in the unsaturated flow conditions that prevail in this system. The concentration of chloride was low enough so that a significant change in surface tension did not take place. Between 95% and 99% of the salt added appeared in the effluent in all runs. Quantities pertaining to the tracer are presented in the tabulated results.

Distribution.—It was necessary to minimize the effect of uneven distribution. The distribution of fluid at the inlet side of the column was therefore established by letting the flow impinge on an inverted funnel such that the flow was distributed in a circular ring on the top of the bed. The area inscribed by the circle was one-half that of the surface area of the bed (Fig. 5). The column containing the 1/2-in. spheres was constructed of clear Lucite so that by visual examination it was possible to observe that within 3 in. from the top of the bed the flow appeared uniformly distributed across the entire cross section.

TABLE 3.—MEDIA AND COLUMN CHARACTERISTICS

Column			Media						
	Diameter, in feet (1)	Height, in feet (2)	Type of spheres (3)	Diameter, in inches (4)	Total Number (5)	Surface Area per sphere, in sq ft (6)	Volume Occupied in cu ft (7)	Specific Surface, in one/ft (8)	Voids, percent of volume (9)
A	0.395	4.1	Glass	0.5	7,850	0.00545	0.503	85.0	41
B	0.667	4.1	Glass	0.75	7,000	0.0123	1.43	60.3	37
C	0.833	5.9	Glass	1.0	6,150	0.0218	3.22	41.6	42
D	3.0	9.0	Porcelain	3.0	4,100	0.196	63.5	12.6	48

Sampling.—Each sample was obtained by collecting the total flow issuing from the column over an increment of time sufficient to collect 50 ml-to-200 ml for analysis. Because the sample constituted the total flow passing the exit cross section for a finite time, it represented an average across the entire cross section, thus minimizing the possible effect of uneven channeling.

OPERATING PROCEDURE

The column was first flushed with water, and a specific, continuous flow rate of top water was applied. Two hours were allowed for establishment of flow equilibrium. The tracer was added, at time $t_o = 0$, to the inlet stream of water above the surface of the bed in such a manner that it would thoroughly mix with the water. Analysis of data secured during trial runs indicated that the apparent mean residence time varied slightly with varying quantities of tracer, all other conditions being equal. This was due to the increased measurability of the tail of the flow-through curve when large quantities of tracer are used. It was decided, therefore, to keep the quantity of tracer applied in constant proportion to the quantity of flow in order to minimize the relative effect. The quantity of tracer added in a run was taken as 0.4 mg of N_aCl per milliliter per minute of flow. The tracer was a 6% solution. The trial runs also indicated the best frequency of sampling, dependent upon the bed and flow rate. At least three samples were obtained for the ascending portion of the flow-through curve (Fig. 1) and from two to four samples to describe the peak (mode). The remaining samples were taken to describe the tail of the curve. The usual volume of sample was from 50 ml-to-200 ml. The chloride content of the sample was determined by the Mercuric Nitrate Method (3) and t_r was evaluated as described. Fig. 2 is a schematic representation of the flow system. Figs. 3 through 8 are photographs of the system. The captions explain these photographs.

RESULTS

Table 4 presents the measured and computed data secured from the four columns during the experiment. The measured flow rates are given in million gallons per acre per day and in cubic feet per square foot per second. The effluent temperature and corresponding kinematic viscosity are listed for each run, as are the quantity of tracer used, the computed mean residence times in seconds and the corresponding dimensionless products (π_1/π_4), π_2 and π_3. The dimensionless products are displayed in graphical form in Fig. 9.

ANALYSIS OF DATA

Examination of the dimensionless plot (Fig. 9) indicates a family of curves as suggested in the theoretical experimental design. Analysis of these curves, however, showed that the curves could be re-arranged into a more descriptive function by dividing the quantity π_2 by the quantity π_3 and plotting this new parameter $Q/S\,\nu$ versus the time function (π_1/π_4). The quantity $Q/S\,\nu$ is a form of Reynolds number, or more correctly, a modified Reynolds number. Fig. 10 is a plot of the data based on this transformation. It shows that the curves have now been collapsed, the three curves representing the glass spheres into one curve, and the curve representing the porcelain spheres into another. It becomes apparent from this plot that we are dealing with two distinct physical traits. The three columns packed with glass spheres display different time characteristics than the column packed with porcelain spheres. Whether the difference in the two systems is caused by a sudden transition in physical action at low specific surface or by the different wetting and roughness characteristics of the media cannot be determined without further research. It is suspected that the latter is true and it is felt that an evaluation of roughness characteristics must be made before these data may be used in practical applications.

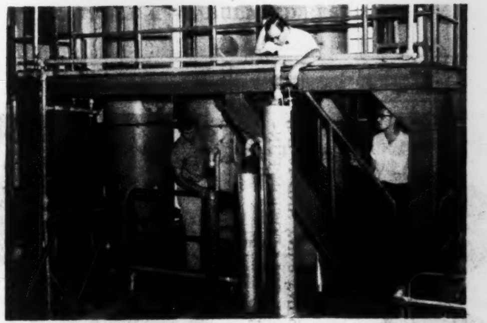

FIG. 3.—OVER-ALL PHYSICAL PLANT

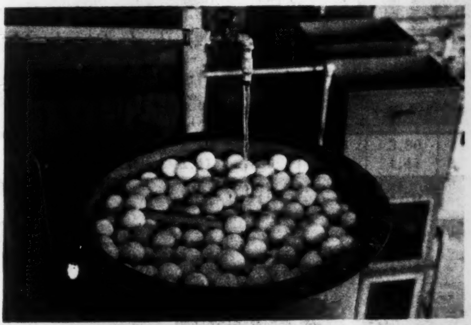

FIG. 4.—TOP OF THE 3 FT DIAMETER COLUMN WITH DISTRIBUTION DEVICE REMOVED

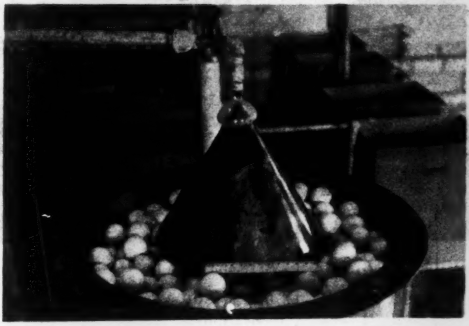

FIG. 5.—TOP OF THE 3 FT COLUMN SHOWING DISTRIBUTION DEVICE

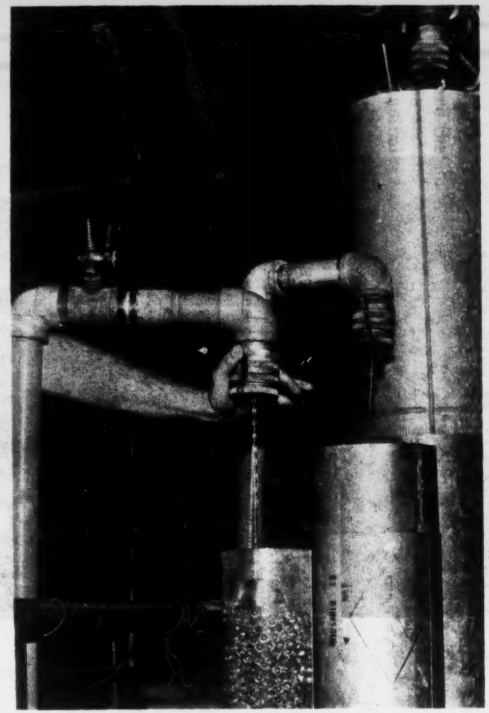

FIG. 6.—THE 5 IN., 8 IN., AND 10 IN., COLUMNS AND ORIFICE DISTRIBUTION

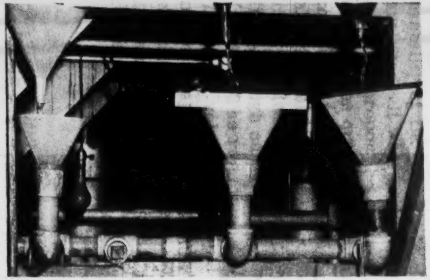

FIG. 7.—EXITS FROM 5 IN., 8 IN., AND 10 IN., COLUMNS WITH COLLECTION FUNNELS IN PLACE

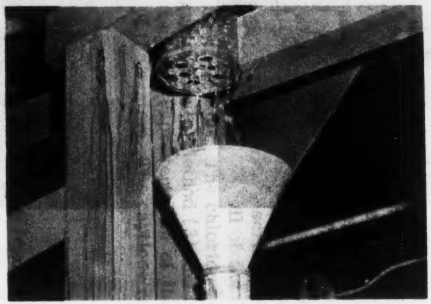

FIG. 8.—EXIT FROM 5 IN. COLUMN SHOWING COLLECTION FUNNEL REMOVED

TABLE 4.—TABULATED RESULTS

Run	Hydraulic Loading Rate		Effluent				Dimensionless Products		
	mgad	ft per sec x 10^3	Temp., °C	ν, x 10^5	N_aCl mg	t_r, in sec	π_1/π_4	π_2	π_3 x 10^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
					Column A				
1	28.8	1.02	15	1.23	90	183	3.28	1.39×10^{-2}	1.43
2	8.87	0.32	22	1.02	31	440	7.40	0.45×10^{-2}	1.26
3	269	9.55	10	1.41	770	29	0.55	1.24×10^{-2}	1.56
4	106	3.76	17	1.17	320	61	1.07	5.21×10^{-2}	1.38
5	9.28	0.33	22	1.02	29	516	8.69	4.78×10^{-3}	1.26
6	28.4	1.01	23	1.00	90	189	3.15	1.47×10^{-2}	1.22
7	105	3.73	14	1.26	336	60	1.08	5.03×10^{-2}	1.45
8	276	9.79	13	1.30	780	28	0.51	1.31×10^{-1}	1.48
9	8.54	0.30	24	0.99	39	558	9.31	4.43×10^{-3}	1.23
10	9.42	0.34	22	1.02	34	492	8.28	4.86×10^{-3}	1.26
					Column B				
1	144	5.14	12	1.34	1200	32	0.59	6.80×10^{-2}	1.07
2	144	5.14	12	1.34	1200	32	0.59	6.80×10^{-2}	1.07
3	4.86	0.17	13	1.14	42	517	9.03	2.42×10^{-3}	0.96
4	14.5	0.52	17	1.19	120	215	3.80	7.16×10^{-3}	0.99
5	58.3	2.08	10	1.41	480	74	1.41	2.70×10^{-2}	1.11
6	151	5.37	10	1.41	1200	33	0.61	6.97×10^{-2}	1.11
7	59.2	2.11	12	1.36	480	68	1.26	2.78×10^{-2}	1.09
8	5.44	0.19	20	1.08	60	480	8.23	2.76×10^{-3}	0.92
9	5.50	0.20	20	1.08	54	480	8.23	2.79×10^{-3}	0.92
10	5.50	0.20	20	1.08	52	474	8.13	2.79×10^{-3}	0.92
11	15.6	0.56	14	1.29	150	214	3.88	7.47×10^{-3}	1.04
					Column C				
1	169	6.01	13	1.30	2400	35	0.44	8.04	7.24
2	66.2	2.35	14	1.28	780	70	0.89	3.16	7.16
3	66.2	2.35	14	1.26	780	69	0.87	3.18	7.11
4	18.5	0.66	15	1.23	264	180	2.24	0.90	6.99
5	169	6.01	13	1.30	1500	33	0.42	8.04	7.24
6	184	0.65	15	1.23	245	182	2.27	0.89	6.99
7	6.16	0.22	18	1.14	102	420	5.09	0.31	6.61
8	6.31	0.22	18	1.14	120	438	5.31	0.31	6.61
					Column D				
1	4.36	1.55×10^{-4}	19	1.13	720	175	1.38	0.22	1.99
2	17.3	6.16×10^{-4}	20	1.08	3000	94	0.74	0.88	1.93
3	107	3.82×10^{-3}	18	1.14	18000	30	0.24	5.34	2.00
4	55.9	1.99×10^{-3}	17	1.17	9110	52	0.42	2.76	2.04
5	57.4	2.04×10^{-3}	17	1.17	9000	55	0.44	2.83	2.04
6	2.06	7.31×10^{-5}	20	1.08	600	279	2.18	0.10	1.93
7	6.81	2.42×10^{-4}	19	1.11	1200	158	1.24	0.34	1.97
8	29.4	1.05×10^{-3}	19	1.11	6000	65	0.51	1.47	1.97
9	14.0	5.00×10^{-4}	19	1.11	3000	90	0.71	0.71	1.97
10	109	3.87×10^{-3}	18	1.14	15000	36	0.287	5.41	2.00
11	9.01	3.21×10^{-4}	18	1.14	1800	120	0.95	0.45	2.00
12	6.63	2.38×10^{-4}	22	1.02	900	140	1.07	0.34	1.86
13	4.62	1.65×10^{-4}	21	1.05	900	183	1.42	0.24	1.90
14	3.39	1.21×10^{-4}	21	1.05	600	210	1.62	0.17	1.90

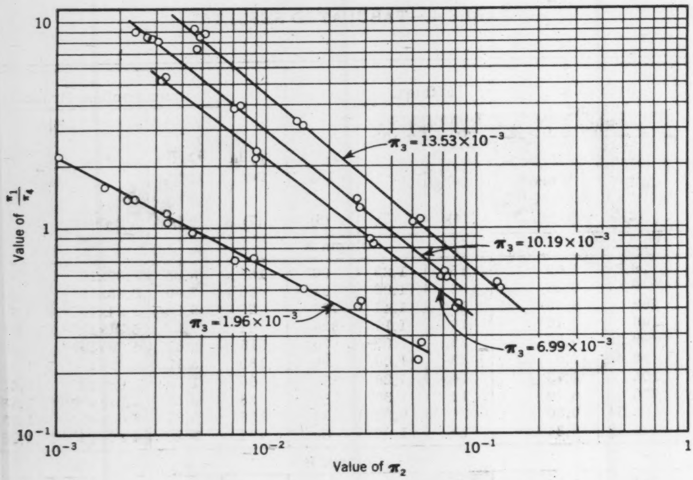
10
1
10^{-1}
Value of $\frac{\pi_1}{\pi_4}$
10^{-3}
10^{-2}
10^{-1}
1
Value of π_2
$\pi_3 = 13.53 \times 10^{-3}$
$\pi_3 = 10.19 \times 10^{-3}$
$\pi_3 = 1.96 \times 10^{-3}$
$\pi_3 = 6.99 \times 10^{-3}$

FIG. 9

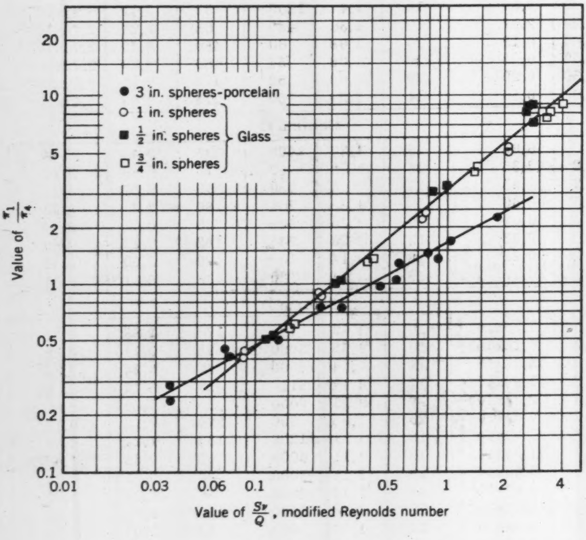
● 3 in. spheres-porcelain
○ 1 in. spheres
■ $\frac{1}{2}$ in. spheres
□ $\frac{3}{4}$ in. spheres
Glass
20
10
5
2
1
0.5
0.2
0.1
Value of $\frac{\pi_1}{\pi_4}$
0.01
0.03
0.06
0.1
0.5
1
2
4
Value of $\frac{Sv}{Q}$, modified Reynolds number

FIG. 10

Theoretical Equation.—Fig. 10 facilitates the graphical evaluation of the constants for a theoretical equation for t_r. Referring to the dimensional analysis and to the above transformation, the form of this equation is

$$\frac{\pi_1}{\pi_4} = f\left(\frac{\pi_3}{\pi_2}\right) \tag{12}$$

or

$$\frac{t_r \, g^{1/3} \, \nu^{1/3}}{H} = f\left(\frac{S \, \nu}{Q}\right) \tag{13a}$$

Because Eq. 12 plots as a straight line on log-log coordinates, it may be written

$$\frac{t_r \, g^{1/3} \, \nu^{1/3}}{H} = K\left(\frac{S \, \nu}{Q}\right)^n \tag{13b}$$

in which K and n are constants. For the glass spheres $K = 3.0$ and $n = 0.83$ and for the porcelain spheres $K = 1.5$ and $n = 0.53$. Thus, for glass spheres

$$t_r = 3.0 \, H \, \frac{\nu^{1/2}}{g^{1/3}} \left(\frac{S}{Q}\right)^{0.83} \tag{14}$$

and for porcelain spheres

$$t_r = 1.5 \, H \, \frac{(\nu)^{0.20}}{g^{1/3}} \left(\frac{S}{Q}\right)^{0.53} \tag{15}$$

ANALYSIS OF RESULTS

Experimental Results - Use of Theoretical Equations.—Eqs. 14 and 15 may be used to predict values of mean residence time for systems similar to the one on which this experiment was conducted. As an example of the possible future use of an equation of this type, suppose it is determined in future studies that the efficiency of a trickling filter is some function of the mean residence time and that one could compute the required mean residence time to yield the desired efficiency. Then, by use of Eq. 14 or its counterpart for a bio-physical system, the loading rate, specific surface and depth may be varied to give the most economical combination providing the desired mean residence time and, thus, the desired purification.

Analysis of the c Versus t Curve For Mean Residence Time.—Three methods of analysis (determination of centroid) of the concentration (c) versus time (t) curve for t_r were investigated.

The Mathematical Method.—The mathematical method requires finding the function relating c to t. Using this relationship Eq. 1 may be integrated and t_r solved for directly. For the curves obtained, the ascending portion of the curve closely approximated a logistic function and the descending portion closely approximated a double exponential-type function. The effluent from the column during the descending portion of the curve may be considered in two phases.

Phase I represents the tracer (or fluid) that constitutes that portion of the total fluid in the column that one considers flowing through. The literature terms this fluid the "operating holdup." Some of the fluid, due to boundary layer theory and the velocity profile in general, does not flow through at the same rate as the "operating holdup." This portion of the flow comes off at a much slower rate and is termed Phase II. Because the concentration of tracer in the effluent should be proportional to the amount remaining in the column, it may be written

$$\frac{dc_1}{dt} = a_1 c_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Solving the differential equation yields

$$c_1 = b_1 e^{a_1 t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which c_1 is concentration due to Phase I. Similarly, for Phase II

$$\frac{dc_2}{dt_2} = a_2 c_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

and

$$c_2 = b_2 e^{a_2 t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

then

$$c = c_1 + c_2 = b_1 e^{a_1 t} + b_2 e^{a_2 t} \quad \ldots\ldots\ldots\ldots (20)$$

Referring to Eq. 1 and assuming a logistic for the ascending portion of the curve,

$$t_r = \frac{\displaystyle\int_{t_1}^{t_2} \frac{t\, c_{t_2}}{1 + xe^{rt}}\, dt + \int_{t_2}^{\infty} \left(b_1 t e^{a_1 t} + b_2 t e^{a_2 t}\right) dt}{\displaystyle\int_{t_1}^{t_2} \frac{c_{t_2}}{1 + x e^{rt}}\, dt + \int_{t_2}^{\infty} \left(b_1 e^{a_1 t} + b_2 e^{a_2 t}\right) dt} \quad \ldots (21)$$

in which t_1 is the time of initial trace, t_2 denotes the modal time, r and x are the logistic parameters, and c_{t_2} refers to the concentration at mode. When integrated, Eq. 21 becomes approximately,

$$t_r = \frac{\dfrac{c_{t_2}}{6}\left(2 t_2^2 - t_1 t_2 - t_1^2\right) - \dfrac{b_1 e^{a_1 t_2}}{a_1^2}\left(c_1 t_2 - 1\right) - \dfrac{b_2 e^{a_2 t_2}}{a_2^2}\left(c_2 t_2 - 1\right)}{c_{t_2}\dfrac{\left(t_2 - t_1\right)}{2} - \dfrac{b_1 e^{a_1 t_2}}{a_1} - \dfrac{b_2 e^{a_2 t_2}}{a_2}} \quad \ldots (22)$$

In the analytical determination, the constants in Eq. 22 may be evaluated by graphical means and may then be solved for t_r.

Graphical Integral Evaluation Method.—In order to evaluate t_r graphically we may plot t versus c and measure the area under the curve. When this method was used in this experiment, the area was determined both by planimeter and Simpson's rule. As previously mentioned this quantity is $\int c\,dt$. (The value of the upper limit being of practical consideration as will be described subsequently.) The quantity $\int c\,t\,dt$ may be evaluated as the area under the c t versus t curve. That is, plot the product of concentration and time versus time and obtain its area. The function t_r is obtained by dividing the latter area by the former.

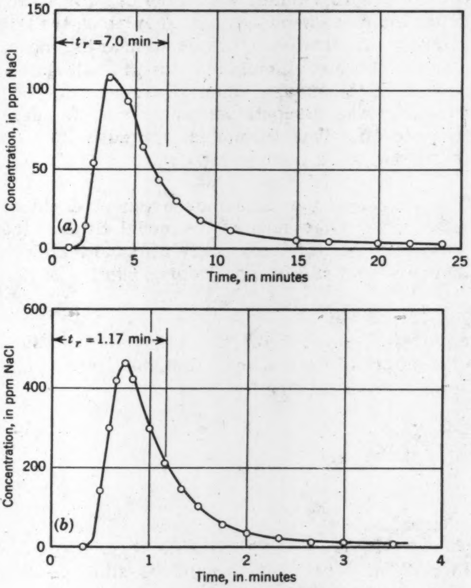

FIG. 11.—ACTUAL FLOW-THROUGH CURVES

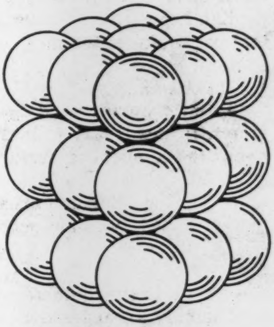

FIG. 12.—RECTANGULAR PACKING

Model Method.—The procedure was to plot concentration versus time on a piece of cardboard and the resulting shape was cut out. The form was then balanced on a knife edge so that the line of contact between the knife edge and the cardboard would be the axis, perpendicular to the abscissa (t axis), passing through the centroid.

Comparison of the Three Methods.—The first fifteen curves obtained using sodium chloride was used for a comparison of the three methods. For any given curve the variation in t_r from one method to another was always less than 5%, with no trend as to one method giving consistently greater or lesser values. The model method was the most convenient and, thus, was the one chosen for routine use.

The "Tail" of the Curve.—As is the case with flow-through measurements of this nature, the skewness of the curve is such that there is a pronounced tail that is very influential on a second moment measurement (see Fig. 11 in which Column C was used with a diameter of 0.833 ft and a depth of 5.9 ft. For Fig. 11(a) Q = 6.16 MGAD = 2.19×10^{-4} fps and 1.7 ml of 6% NaCl solution was applied. For Fig. 11(b) Q = 66.2 MGAD = 2.35×10^{-3} fps and 13 ml of 6% NaCl solution was applied). Theoretically, the mean resistance time would be infinite if only one minute particle of tracer remain within the bed for an infinite time. To add to the difficulty, the measurements of tracer along the tail are obviously less accurate percentagewise than measurements near the mode. The method used in computing t_r (by the model method) aided in alleviating the difficulty. When the analytical error in measurement of tracer exceeded 25% of the concentration, measurement was discontinued and a smooth curve extrapolated to give a reasonable die-off. This technique worked quite well because replicate runs gave results that were in close agreement as indicated in the tabulated results.

The influence of the tail of the curve on the time measurement could be minimized by using another criterion for time such as the modal time or the median time. It was assumed, however, that for later correlation with biological parameters, the mean residence time would be most representative of this system.

The Media.

Specific Surface and Sphere Packing.—In the computation of specific surface it is necessary to include the effect of void space. One may mistakenly compute the specific surface of any sphere packing as

$$\frac{\text{Surface area of one sphere}}{\text{Volume occupied by the sphere}} = \frac{\pi d^2}{\frac{\pi d^3}{6}} = \frac{6}{d} \quad \ldots\ldots\ldots \quad (23)$$

in which d is the diameter of spheres.

The only arrangement of spheres that Eq. 23 is applicable to, is a rectangular arrangement shown in Fig. 12. Because of the many possible packing arrangements, the specific surface must be computed as

$$S = \frac{\text{(Number of spheres) (surface area of one sphere)}}{\text{Volume occupied}} \ldots \quad (24)$$

Then, if the void space is known

$$S = N \pi d^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (25)$$

in which N refers to the number of spheres per cubic foot. But

$$N = \frac{(1 - V)\ 6}{\pi d^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (26)$$

in which V is the fraction of voids. Therefore,

$$S = \frac{(1 - V)\, 6\, \pi\, d^2}{\pi\, d^3} = \frac{6\, (1 - V)}{d} \quad \ldots\ldots\ldots\ldots \quad (27)$$

Table 3 gives the column and media characteristics.

Wall Effect and Scale-Up.—Because of the possibility of "wall effect" (part of the flow short-circuiting down the walls of the column and thereby interfering with the usual flow process) it would be desirable to keep the ratio of the specific surface of the walls of the column to the specific surface of the media as small as practical and constant from column to column. Keeping the ratio constant from column to column in no way eliminates "wall effect," however, it should provide the same degree of influence in all beds.

"Wall effect" should tend to decrease the apparent mean time, because the flow down a straight vertical wall is less tortuous than around, and over the media. It might be expected, therefore, that the results of this experiment tend to give values of t_r that are smaller than if there was no "wall effect."

TABLE 5.—RATIOS PERTINENT TO WALL EFFECT

	Column Diameter, in feet (1)	Specific Surface in square feet per cubic foot		Percent, S wall	Diameter Column
		Column wall (2)	Media (3)	S media (4)	Diameter media (5)
A	0.395	10.1	85.0	11.9	9.5:1
B	0.667	6.0	60.3	9.9	10.7:1
C	0.833	4.8	41.6	11.5	10.0:1
D	3.00	1.33	12.6	10.5	12.0:1

Previous work (4) indicates, however, that if the ratio of tower to packing diameter is equal to or exceeds eight to one, "wall effect" becomes negligible and scale-up difficulties are minimized. Because the ratio of column diameter to media diameter herein exceeded eight-to-one in all cases (Table 5) it would be expected that the error in the value of t_r due to "wall effect" would be small and thus make these data useful in prediction of t_r when applied to a large-scale setup.

Theoretical Derivation Based on Physical Laws.—W. E. Howland (5) derived an equation for the time of flow over a single sphere assuming that the liquid was applied at the very top, spread out over the entire sphere, and left at the very bottom. Actually, as Howland recognizes, and as has been confirmed by visual examination, these assumptions are open to questions. Unless a wetting agent is added, the liquid tends to channel instead of spreading over the sphere. Where the liquid starts on a sphere and where it leaves is a function of the places of contact with adjacent spheres. Because it is believed that these assumptions distract from the true physical situation no correlation will be drawn between the theoretically assumed and the actual, as they are distinctly two different cases. It should be pointed out, however, that if a slime is developed on the entire sphere the fluid will spread over the entire sphere and the error due to that assumption is minimized. The effect of slime itself, however, introduces quite a different factor for consideration, and will be evaluated in future experiments. Howland's equation is

$$T_s = 2.6 \left(\frac{3\,\nu}{g}\right)^{1/3} (2\,\pi)^{2/3} \frac{r^{5/3}}{(Q')^{2/3}} \quad \ldots\ldots\ldots \quad (28)$$

in which T_s is the time of flow around one sphere from top to bottom, r refers to the radius of sphere, and Q' denotes the total flow on one sphere. Transforming this equation into the notation used herein (See Appendix I) we obtain

$$T_r = 1.3\ H \left(\frac{3\,\nu}{g}\right)^{1/3} \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots\ldots \quad (29)$$

Assuming a fluid temperature of 15° C and evaluating the known quantities, Eq. 29 becomes

$$\frac{T_r}{H} = 0.0136 \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots\ldots\ldots \quad (30)$$

Fig. 13 is a plot of Eq. 30 with the experimental data indicated for comparison. Referring to Fig. 13, we are again aware of two distinct regimes. The set of data from the three columns containing the glass spheres gives different results from that set of data of the procelain spheres. The difference may be attributed to many factors, and, to reiterate, they include (1) different wetting effect between glass and porcelain, and (2) different roughness characteristic between glass and procelain.

Again, the important conclusion concerning the facts previously cited is that, with two different physical systems two different sets of results were obtained; with the same physical system the same set of results based on theoretical parameters were obtained. It is important to consider this fact when applying these data to different physical systems.

Past Time Studies and Relationship to Chemical Industrial Processes.— Various attempts to measure the time of flow through a trickling filter have been made (6),(7),(8),(9),(10),(11),(12),(13),(14),(15),(16).

The only full scale attempt using clean media seems to have been done by W. Clifford (7) in 1907. Through uncertainties as to periodicity of dosing the specific surface of his media and the porosity of his media we are prevented from drawing any definitive correlation between his data and the experimental data presented herein. Nevertheless, referring to Fig. 14, observe that, in light of his data, the experimental curves may be extrapolated into the lower loading ranges without resulting in an appreciable error.

The results of the other experiments are applicable, in general, only to the particular filter on which the experiment was run. No general equations or formulations, as of the type developed here, seem to have evolved.

A review of the literature reveals that flow through granular media has application in many fields. The literature of the chemical industry dealing with

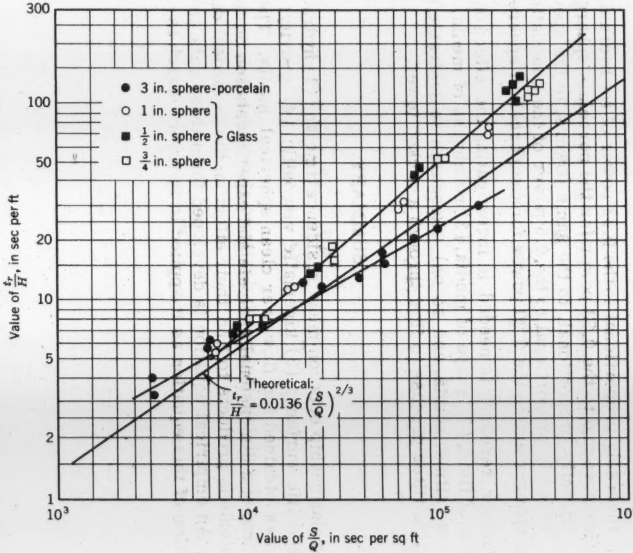

FIG. 13.—THEORETICAL CURVE AND EXPERIMENTAL DATA

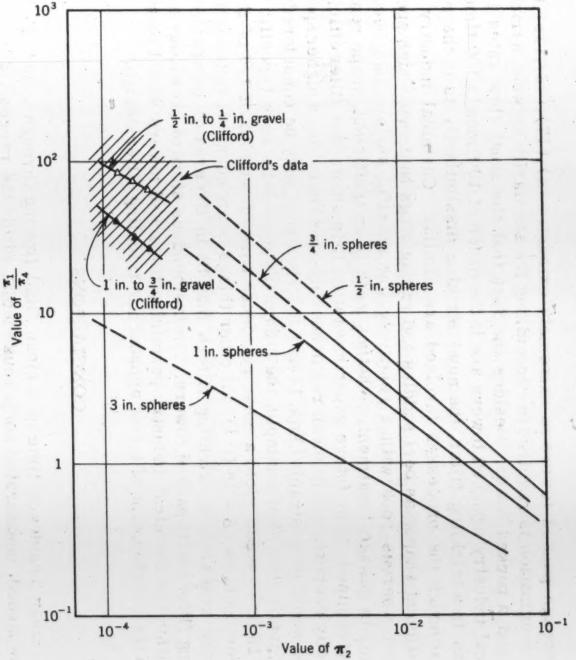

FIG. 14.—COMPARISON OF EXPERIMENTAL DATA WITH THAT OF CLIFFORD

flow through packed towers is extremely extensive (4),(17),(18),(19),(20). Little imagination is required in recognizing the similarity between a trickling filter and a packed tower. Besides the fact that the usual flow rates in the chemical industry's packed towers are in the order of 10 times to 100 times as great as in a trickling filter, the most striking dissimilarity is in the media used, although the processes involved are similar. Chemical industry uses such efficient shapes as berl saddles and rashig rings to provide high specific surfaces, whereas rock with a relatively low specific surface has been the mainstay in sewage treatment. Whether rock is an inefficient shape remains to be determined. If by future experimentation it is shown that filter efficiency is directly proportional to mean residence time, and therefore a higher specific surface would be desirable (Eqs. 14 and 15), then rock may be considered relatively inefficient. For example, the commonly used 3-in. rock (specific surface = 15 ft^{-1}) would yield a mean residence time of only about 1/4 of that of 1-in. berl saddles ($S = 76\ ft^{-1}$), all other things being equal. It is thought that important research on trickling filters may be in the area of comparison of trickling filters with packed towers. The tremendous store of knowledge available within the chemical industry pertaining to packed towers would be available to the development of a new approach to trickling filter design.

CONCLUSIONS

The mean residence time of a clean fluid flowing through a bed of clean, loosely-packed, unsaturated nonporous media, within the ranges stated herein is as follows:

1. Inversely proportional to the 0.83 power of the hydraulic loading rate for glass spheres and to the 0.53 power for the porcelain spheres;
2. Directly proportional, to the same powers, to the specific surface; and
3. Directly proportional to the 0.5 power of the kinematic viscosity for the glass spheres and to the 0.20 power for the porcelain spheres.

Further research is needed in investigating the effect on t_r of (1) surface tension of fluid, (2) shape of media, (3) roughness of media, and (4) biological entities (slime, solids, and so on). Research is desirable in correlating the trickling filter process to flow through a packed tower.

SUMMARY

A relationship between mean residence time and (1) hydraulic loading rates; (2) specific surface; (3) kinematic viscosity; (4) gravity; and (5) depth of bed, was obtained for flow over clean spherical media. The approach was in the form of a dimensional analysis and experimentation on the independent dimensionless products to find their effect on the dependent dimensionless product. An empirical equation is developed form these data and the possible application of this equation as it applied to future research is presented.

APPENDIX I

Referring to Eq. 28 (Howland's equation), let N_h denote the number of spheres per square foot of horizontal section, and N refer to the number of spheres per cubic foot, (all computations are based on an assumed rectangular packing). Then,

$$N = N_h \frac{1}{2\,r} \quad \dots\dots\dots\dots\dots\dots \quad (31)$$

$$\text{Specific surface} = N\, 4\,\pi\, r^2 = S \quad \dots\dots\dots\dots \quad (32a)$$

Then,

$$N = \frac{S}{4\,\pi\, r^2} \quad \dots\dots\dots\dots\dots\dots \quad (32b)$$

and

$$\frac{S}{4\,\pi\, r^2} = \frac{N_h}{2\,r},\; N_h = \frac{S}{2\,\pi\, r} \quad -\dots\dots\dots\dots \quad (33)$$

Letting Q be the total flow per square foot of column cross section,

$$Q = Q'\; N_h,\; Q' = \frac{Q}{N_h} = \frac{2\,\pi\, r\, Q}{S} \quad \dots\dots\dots\dots \quad (34)$$

$$T_s = 2.6\left(\frac{3\,\nu}{g}\right)^{1/3} (2\,\pi)^{2/3} \frac{r^{5/3}}{(2\,\pi)^{2/3}} \; \frac{S^{2/3}}{(r)^{2/3}\, Q^{2/3}} \quad \dots\dots \quad (35)$$

and

$$T_s = 2.6\left(\frac{3\,\nu}{g}\right)^{1/3} \frac{r\, S^{2/3}}{Q^{2/3}} \quad \dots\dots\dots\dots\dots\dots \quad (36)$$

If N_v is the number of spheres in vertical direction, and H refers to the depth of packing (of the column)

$$T_s\, N_v = T_r \quad \dots\dots\dots\dots\dots\dots \quad (37a)$$

$$N_v = \frac{H}{2\,r} \quad \dots\dots\dots\dots\dots\dots \quad (37b)$$

and

$$T_s \frac{H}{2\,r} = T_r \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(37c)}$$

Then

$$T_r = 2.6\ H \left(\frac{3\ \nu}{g}\right)^{1/3}\ 1/2 \times \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots\ldots \text{(38a)}$$

or

$$T_r = 1.3\ H \left(\frac{3\ \nu}{g}\right)^{1/3} \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots\ldots\ldots \text{(38b)}$$

Assuming a temperature of 15°C, and a value of $\nu = 1.23 \times 10^{-5}$ sq ft per sec,

$$\frac{T_r}{H} = 1.3\ \frac{\left(3 \times 1.23 \times 10^{-5}\right)^{1/3}}{32.2} \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots \text{(39a)}$$

$$\frac{T_r}{H} = 1.3\ \left(1.147 \times 10^{-6}\right)^{1/3} \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots\ldots \text{(39b)}$$

and

$$\frac{T_r}{H} = 1.3\ (1.0466)\ \left(10^{-2}\right) \left(\frac{S}{Q}\right)^{2/3} \quad \ldots\ldots\ldots\ldots \text{(39c)}$$

from which Eq. 30 can be obtained.

APPENDIX II.—NOTATION

A = surface area of bed;

c = concentration;

c_{t_2} = concentration at mode;

d = diameter of spheres;

g = acceleration of gravity;

H = depth of bed or column as noted;

N = spheres per cubic foot;

N_h = number of spheres per square foot of horizontal section;

N_v = number of spheres in the vertical direction for depth H;

Q_1 = rate of fluid application;

Q = hydraulic loading rate;

Q' = total flow on one sphere;
R = roughness of media;
r = radius of sphere;
r and x = logistic parameters;
S = specific surface of media;
S_1 = size of media;
S_2 = packing density of media;
S_3 = shape of media;
T_s = time of flow around one sphere from top to bottom;
t = time;
t_r = mean residence time;
t_1 = time of initial trace;
t_2 = modal time;
V = mean fraction of voids;
μ = viscosity of fluid;
ν = kinematic viscosity of fluid;
ρ = density of fluid; and
σ = surface tension of media.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3245

LABORATORY RESEARCH ON INTERCEPTOR DRAINS

By Jack Keller,[1] and A. R. Robinson,[2] M. ASCE

With Discussion by Messrs. M. Maasland; William F. Long; William W. Donnan; Herman Bouwer; Jan Van Schilfgaarde; R. William Nelson; John G. Sutton; and A. R. Robinson

SYNOPSIS

The results of a large scale model study concerned with the design of interceptor drains are presented. Previously developed analytical relationships for determining the shape of the resultant water table drawdown curve are analyzed. Relationships are developed for estimating the flow from the drains. Examples are given to illustrate the use of the material.

INTRODUCTION

There have been many studies made of drainage problems concerned with relief or grid-type drains. However, there is little information available on the hydraulics of interceptor drainage. Interception of ground water flowing laterally from a source which may be outside the affected area is the common problem. The drain must be located for maximum benefit to the problem area. The variables to be considered are depth, length, and size of drains, and the location relative to the problem area and source of seepage.

Note.—Published essentially as printed here, in September, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2154. Positions and titles given are those in effect when the paper was approved for publication in Transactions.

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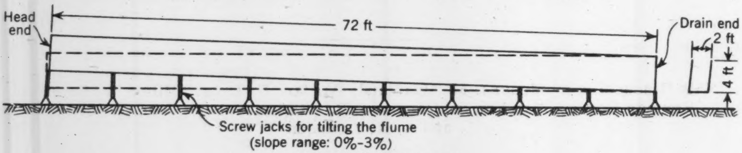
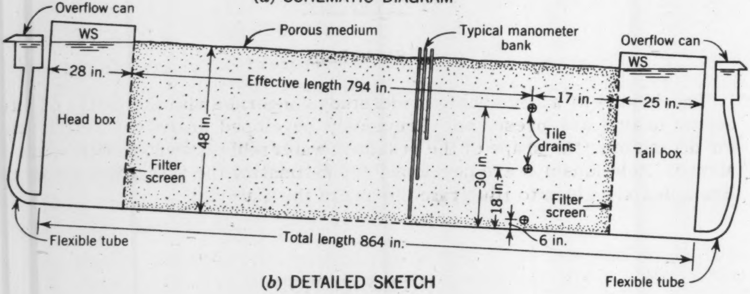
[2] Agric. Engr., Western Soil and Water Mgt. Research Branch, Agric. Research Service, Colorado Agric. Experiment Sta., Fort Collins, Colo.

The purpose of the study reported herein was to investigate a type of interceptor drain[3] in which there was a source of seepage at some finite distance from the projected location of the drain. In the experiment an impermeable boundary with constant slope existed at a measurable distance below the ground surface. The source of seepage was such that the water depth at the source point would remain unchanged after drainage. The factors which were investigated were the flow into the drain installation and the resulting drawdown curve.

The experiment was designed to establish the relationship between the pertinent variables and to obtain data for comparison with the results from other investigators. A check on the accuracy of theoretically derived relationships was one of the objectives. Dimensional analysis was used to relate the variables for a more systematic study.


Head end
72 ft
Drain end
2 ft
4 ft
Screw jacks for tilting the flume (slope range: 0%-3%)

(a) SCHEMATIC DIAGRAM


Overflow can
WS
Porous medium
Typical manometer bank
Overflow can
WS
28 in.
Effective length 794 in.
17 in.
25 in.
Head box
48 in.
Tile drains
Tail box
Filter screen
30 in.
18 in.
Filter screen
Flexible tube
Total length 864 in.
6 in.
Flexible tube

(b) DETAILED SKETCH

FIG. 1.—LAYOUT OF TILTING FLUME

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference, in the Appendix.

EQUIPMENT

The study was conducted in the Hydraulics Laboratory at Colorado State University, Fort Collins, Colo. utilizing a large tilting flume which is shown schematically in Fig. 1. The flume was 70 ft long, 2 ft wide, and 4 ft high and could be adjusted for slope from horizontal to 3%. The flume was filled with sand to a depth of 44 in. A head and tail box with adjustable overflow devices were provided to control ground water levels. Tile drains were placed at three levels near the downstream end of the flume with an additional tile drain near the mid-point. Banks of manometers connected by plastic tubing to piezometers placed at intervals along the flume were used to determine the ground water profile. The outflow from the drains was weighed to determine the discharge.

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[3] "Model Studies of Interceptor Drains," by Jack Keller, thesis presented to Colorado State Univ., at Fort Collins, in 1955, in partial fulfilment of the requirements for the degree of Master of Science.

The material used in the study was a decomposed granitic sand having a mean size of 0.107 in. (2.72 mm) and a uniformity coefficient of 2.0. Hydraulic conductivity was determined to be 0.038 ft per second and a capillary rise of 1.5 in. was indicated. The material was compacted to uniform density as indicated by conductivity measurements made with variable depths of ground water in the flume. The porosity of the in-place material was determined to be 36.8% and the specific yield was 25.7%.

PROCEDURE

The procedure used was such that the drawdown curve being investigated followed a higher drawdown curve. A minimum of 3 hr was allowed after a given set of boundary conditions was imposed in order that equilibrium be established. With the head water depth held constant, one of a series of drainage conditions was imposed either by opening the valve for any one of the tile drains or by adjusting the level of the tail water. The tail box actually simulated an open, interceptor drainage ditch.

Head-water depths were varied from 8 in. to 40 in. Various tile drains were operated with each constant head water depth. The slope was varied in 1/2% increments from 0% to 3%.

THEORETICAL ANALYSES

Flow into Drain.—At this point it is necessary to show, by dimensional analysis, the relationships that exist for the flow. For a simple system with no drain the variables for a two-dimensional system may be expressed by

$$q_o = \phi_1 \ (S, K, H) \dots\dots\dots\dots\dots\dots (1)$$

in which q_o is the flow per unit width, K denotes the hydraulic conductivity and H refers to the depth of ground water above an impermeable boundary of slope S. Choosing K and H as repeating variables yields

$$\frac{q_o}{K\,H} = \phi_2 \ (S) \dots\dots\dots\dots\dots\dots (2)$$

In general, one must determine ϕ_2 by experimentation. However, from Darcy's law

$$q_o = K\,H\,S \dots\dots\dots\dots\dots\dots (3)$$

which leads to the conclusion that $\phi_2 (S) = S$.

When a drain is installed as in Fig. 2, the relationship that exists is

$$q_d = \phi_3 \, (S, K, r, L, H, h) \ldots\ldots\ldots\ldots\ldots \quad (4)$$

in which q_d is the resulting flow in the drain, r represents the radius of the tile, L denotes the distance from the tile line to the source of seepage, and h is the distance above the barrier layer to the drain. The remaining variables have the same meaning as in Eq. 1. Eq. 4 may be rewritten in more usable terms as

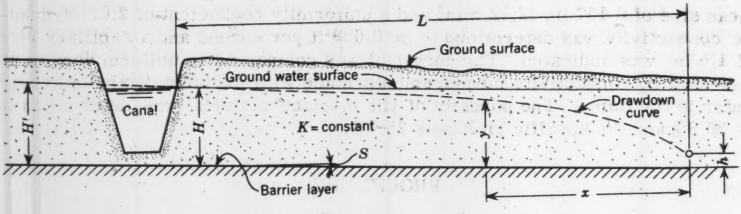

FIG. 2.—LAYOUT OF A TILE INTERCEPTOR DRAIN

$$q_d = \phi_4 \left[S, K, r, (H + S\,L), H, h \right] \ldots\ldots\ldots\ldots \quad (5)$$

Choosing K and H as repeating variables and combining yields

$$\frac{q_d}{H\,K} = \phi_5 \left[\frac{r}{H}, \frac{(H + S\,L)}{H}, \frac{h}{H}, S \right] \ldots\ldots\ldots\ldots \quad (6)$$

One may infer from Eq. 6 that $q_d \neq q_o$ in Eq. 1 because additional parameters are involved. Dividing $q_d/K\,H$ by the dimensionless parameter S yields

$$\frac{q_d}{K\,H\,S} = \phi_6 \left[\frac{r}{H}, \frac{(H + S\,L)}{H}, \frac{h}{H}, S \right] \ldots\ldots\ldots\ldots \quad (7)$$

Darcy's equation yields $K\,H\,S = q_o$, uniquely relating K and S. By substituting q_o for K H S, elimination of S as a separate variable appears possible, thus

$$\frac{q_d}{q_o} = \phi_7 \left[\frac{r}{H}, \frac{(H + S\,L)}{H}, \frac{h}{H} \right] \ldots\ldots\ldots\ldots \quad (8)$$

The parameter r/H may be dropped since it is not important provided the tile is large enough to carry the required flow or an open drain is used. The converse of the reciprocal of (H + S L)/H was found to be more useful for showing the relationships; as a result Eq. 8 may be changed to

$$\frac{q_d}{q_o} = \phi_8 \left[\frac{S\,L}{(H + S\,L)}, \frac{h}{H} \right] \ldots\ldots\ldots\ldots \quad (9)$$

Eq. 9 is the flow analysis of the type of interceptor drain studied.

Shape of the Drawdown Curve.—An analysis similar to that for the flow analysis was made for the shape of the drawdown curve. However, this analysis was found to be difficult to handle because of the number of parameters involved. It was found that a previous theoretically derived relationship was applicable. One purpose of this study was to check this equation using model techniques. This equation which was developed by R. E. Glover, F. ASCE and presented by William W. Donnan,[4] F. ASCE is

$$x = \frac{H \log_e \frac{(H - h)}{(H - y)} - (y - h)}{S} \qquad (10)$$

in which x and y are the coordinates of any point on the drawdown curve as shown by Fig. 2 and the remaining variables have the same meaning as in Eqs. 1 and 4. This equation yields infinite values of x when S = 0 or y = H. The assumption was made that the flow remains constant before and after drainage.

ANALYSES OF DATA

If all variables had been given proper consideration in the dimensional analyses, the resulting parameters would be functionally related. The relationship would be demonstrated by the alinement of experimental data when plotted. The value of dimensional analysis is reflected by the degree to which the data are summarized, condensed or generalized by plotting in the dimensionless form.

Flow into Drain.—In the analysis of the flow data, three assumptions were made: The first was that the capillary flow was negligible; the second assumption was that the tile drains were completely effective, that is there was no bypass flow over the drains; the third was that an open ditch interceptor was simulated by using the tail box to intercept all the flow.

Fig. 3 is a plot of the dimensionless parameters given in Eq. 9. Points are labeled with numbers representing values of $h/H \times 100$. The parameter h/H is a ratio of height of drain above the impermeable boundary to the depth of water-bearing stratum. A value of h/H equal to zero indicated the drain was placed on the barrier layer. The parameter $SL/(H + S L)$ shows the relationship of energy in a system because of slope to that due to slope and depth. This approaches a value of one for a great length or small values of H and decreases as the distance from the source to the drain decreases or H increases. The alinement of points in Fig. 3 indicates that all the factors influencing the problem had been considered. By using a dimensionless plof, the results can be used for the solution of problems of any size.

Shape of Drawdown Curve.—The comparison of plottings of Eq. 10 with experimental data is shown in Fig. 4. The results of two tests are shown, one of which could be considered a relatively short system ($H = 40$, $L = 811$, $h = 0.0$) and the other relatively long ($H = 13.3$, $L = 794$, $h = 5.0$). For both systems $S = 0.03$. For the short system there was a considerable difference between the observed drawdown curve and the computed curve. Some adjustment was therefore necessary in Eq. 10. This was accomplished by computing a new

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[4] "Drainage of Agricultural Lands Using Interceptor Lines," by W. W. Donnan, U. S. Dept. of Agric., SCS, Div. of Irrigation, 1953, p. 13.

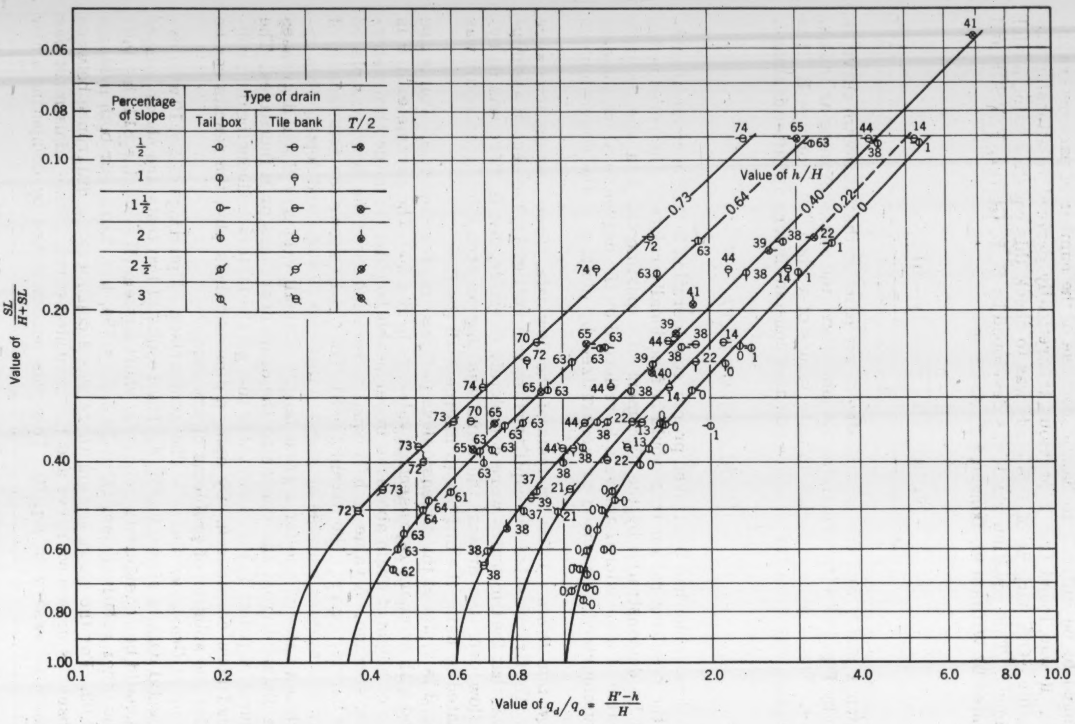

FIG. 3.—DISCHARGE OF INTERCEPTOR DRAINS FOR SLOPES GREATER THAN ZERO

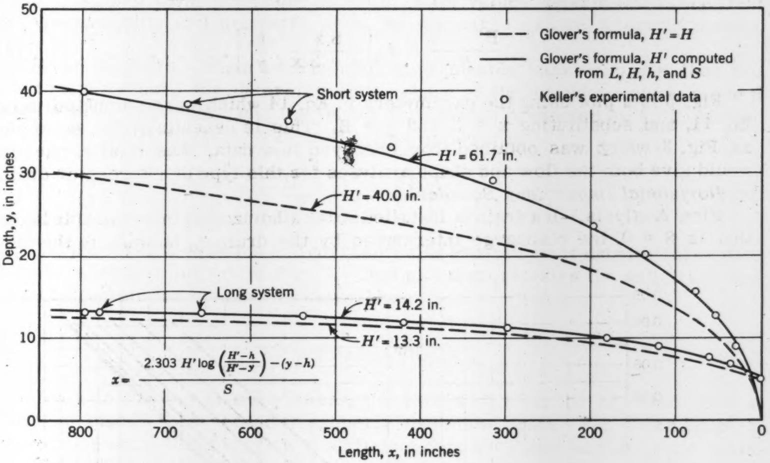

FIG. 4.—COMPARISON OF OBSERVED DRAWDOWN CURVES WITH GLOVER'S FORMULA FOR SLOPES GREATER THAN ZERO

value for H in the equation which will hereafter be called H' and was done by substituting x = L and y = H back into the equation and solving for H'. Using this new value for H', Eq. 10 checked with observed data as shown in Fig. 4. Eq. 10 then becomes

$$x = \frac{H' \operatorname{Log}_e \frac{(H' - h)}{(H' - y)} - (y - h)}{S} \quad \ldots\ldots\ldots\ldots(11)$$

At this point it was noted that the shape analysis was related to the flow analysis. The following relationships will illustrate this statement:

$$\frac{q_d}{q_o} = \frac{q_t - q_b}{q_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(12)$$

and

$$\frac{q_d}{q_o} = \frac{H' Ks - hKs}{HKs} = \frac{H' - h}{H} \quad \ldots\ldots\ldots\ldots(13)$$

in which q_t is total flow in a system with drain and q_b is bypass flow past the drain. This relationship is shown in Fig. 3.

By substituting $(H' - h)/H$ for q_d/q_o, y for H and x for L into Eq. 9, which is the flow analysis, a related shape analysis is produced as

$$\frac{H' - h}{y} = \phi_9\left[\frac{S\,x}{y + S\,x}, \frac{h}{y}\right] \ldots\ldots\ldots\ldots\ldots (14)$$

Fig. 5 is a plot using the parameters in Eq. 14 which were computed using Eq. 11, and substituting $x = L$ and $y = H$. This is essentially the same plot as Fig. 3 which was obtained from observed flow data. Essentially, one plot would give both the flow and shape analyses for this type of interceptor drain.

Horizontal Impervious Boundaries.—

Flow Analysis.—If a drain is installed above a horizontal impermeable layer, that is $S = 0$, the discharge intercepted by the drain q_d is equal to the total flow q_t since there is no bypass flow q_b and the general functional relationship is

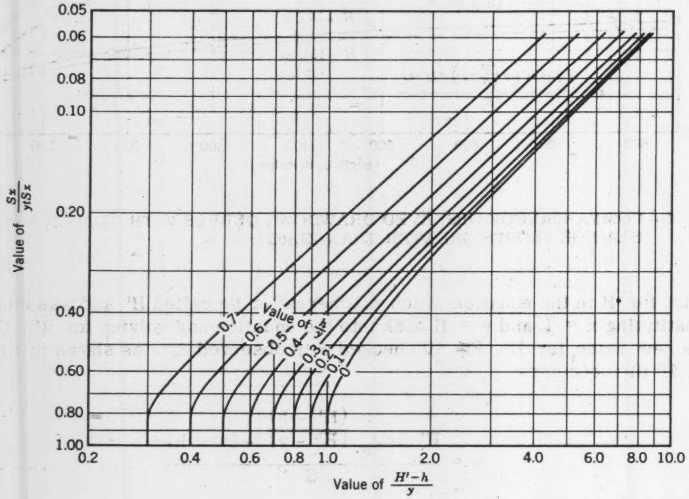

FIG. 5.—SHAPE OF DRAWDOWN CURVES FOR INTERCEPTOR DRAINS

$$q_d = \phi_{10}\ (K, L, H, h, r) \ldots\ldots\ldots\ldots\ldots (15)$$

with the terms as previously defined.

Choosing K and H as repeating variables and combining yields

$$\frac{q_d}{KH} = \phi_{11}\left(\frac{L}{H}, \frac{h}{H}\right) \ldots\ldots\ldots\ldots\ldots (16)$$

in which r/H is again regarded as being insignificant.

The validity of Eq. 16 is demonstrated by Fig. 6 which shows the relationship computed from experimental data. With values determined in the field for permeability and geometry, then the discharge can be determined using Fig. 6.

Shape Analysis.—When a horizontal, impermeable barrier exists, then Eq. 11 cannot be used for determining the shape of the drawdown curve since S equals zero. For this case, the Dupuit formula as presented by F. T. Mavis, F. ASCE, and T. P. Tsui[5] was used. This equation is

$$\frac{H^2 - y^2}{H^2 - h^2} = \left(\frac{L - x}{L}\right)^n \qquad (17)$$

The plot of this equation and corresponding experimental data for three different cases are shown in Fig. 7. This plot demonstrates the validity of the Dupuit equation for the case of zero slope.

APPLICATIONS

Flow into Drain.—In many cases, the flow which can be expected after a drain is installed is needed to properly design the drain. Fig. 3 can be used to make an estimate of the flow if sufficient data are gathered before the drain is installed. For this determination, it is necessary to (1) make an estimate of distance, L, which is the length from the drain to the seepage source or to a point where the installation of the drain will not change the elevation of the water table to any appreciable extent; (2) determine thickness of the water-bearing aquifer; (3) find general slope of the water table or impermeable layer before drainage; and (4) determine average hydraulic conductivity.

As a practical example let it be assumed that the length, L, is 200 ft, the water-bearing stratum is 10 ft thick, H, overlying an impermeable boundary of slope 0.01, S, with a tile drain installed 4 ft, h, above the barrier layer. Solving for the known variables yields

$$\frac{S\,L}{(H + S\,L)} = 0.17 \text{ and } \frac{h}{H} = 0.4$$

From Fig. 3

$$\frac{q_d}{q_o} = 2.6 \text{ or } q_d = 2.6\, q_o$$

The discharge in the drain, q_d, would be 2.6 times the flow per linear foot which was occurring before drainage, q_o. If the hydraulic conductivity is known, the actual drain discharge can be computed. Assuming a conductivity, K, of 0.0001 ft per sec (4.3 in. per hr)

$$q_o = H\,K\,S = (10)(0.0001)(0.01) = 0.00001 \text{ cfs per linear ft}$$

---

[5] "Percolation and Capillary Movements of Water Through Sand Prisms," by F. T. Mavis and T. P. Tsui, Univ. of Iowa, Studies in Engineering, Bulletin 18, April, 1939, pp. 1-31.

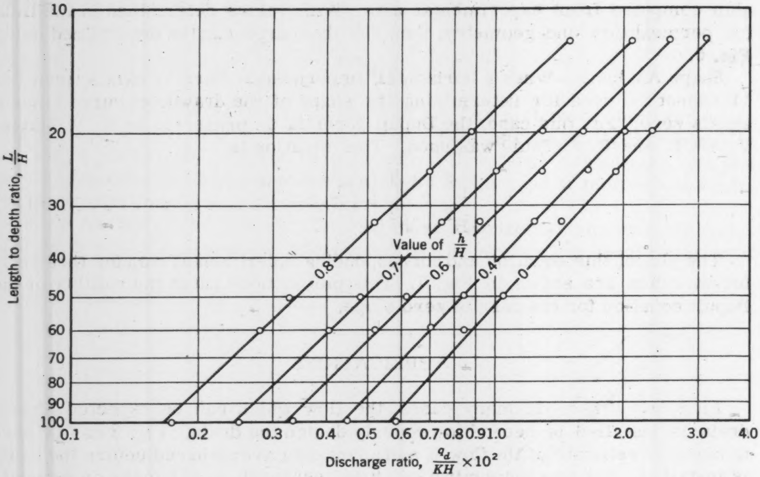

FIG. 6.—DISCHARGE AS A FUNCTION OF DISTANCE TO SOURCE FOR SLOPE EQUAL TO ZERO

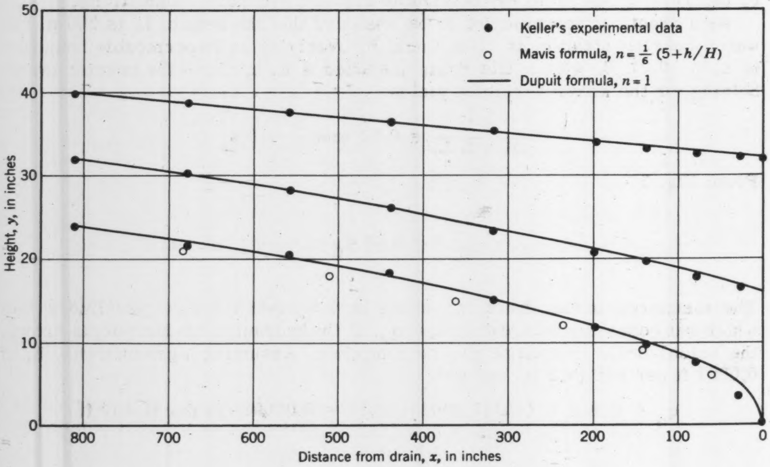

FIG. 7.—COMPARISON OF OBSERVED AND COMPUTED DRAWDOWN CURVES FOR SLOPE EQUAL TO ZERO

or

$$q_o = 4.5 \text{ gpm per 1,000 linear ft}$$

since

$$q_d = 2.6\, q_o$$

$$q_d = 11.7 \text{ gpm per 1,000 linear ft of drain}$$

In the foregoing example, the measurement of four variables is necessary in order to determine the resultant drain flow. The length L is probably the most difficult to determine. For the case of interception of canal seepage, this is simply the distance from the canal to proposed drain. When the ground water is flowing laterally from some undefined source at a great distance from the problem area, then the selection of L is also simplified. For large values of L the parameter $S\,L/(H + S\,L)$ approaches one (Fig. 3) so that the drain will merely remove a percentage of the total flow equal to the ratio of the installed depth below the original water table to the original depth of ground water flow.

The conditions under which interceptor drains are used may often be more complex than that of the simple steady-state flow systems just described and the selection of L is more a matter of judgment. Probably the most common problem is one in which the waterlogged condition is due to the operations of an uphill farmer who applies water at such times as his crops require. Each time water is applied a transient condition is initiated because of percolation to the water table. Since the solution presented is only for a steady state condition, it is necessary to adapt this method in order to estimate the flow which the drain will be required to handle.

The worst possible condition which the uphill irrigator could cause would be if the water table was maintained permanently at the ground surface at his field boundary. A steady state drain flow could then be determined, as in the foregoing example, using the distance from the proposed drain location to the uphill field boundary as L and the depth from the ground surface at the field boundary to the lower confining layer as H. The flow so determined would be larger than would actually occur except immediately after the drain was installed.

Shape of the Drawdown Curve.—The shape of the drawdown curve resulting from the installation of an interceptor drain can be determined using Figs. 3 and 5. As a practical problem, suppose that the original depth of water-bearing strata, H, was 10 ft, the drain was 5 ft above the barrier layer, h, which had a slope, S, of 0.02 and was installed at a distance, L, of 500 ft from the known source of seepage. Then

$$\frac{h}{H} = 0.5 \qquad \frac{S\,L}{(H + S\,L)} = 0.5$$

from Fig. 3

$$\frac{(H' - h)}{H} = 0.69$$

$$H' = 11.9$$

The problem is to determine the distance, x, from the drain that the ground water would be lowered 2.5 ft from its original level. Then

$$y = 10.0 - 2.5 = 7.5 \text{ ft}$$

$$\frac{h}{y} = \frac{5}{7.5} = 0.67$$

$$\frac{(H' - h)}{y} = 0.92$$

from Fig. 5

$$\frac{S x}{(y + S x)} = 0.27$$

$$x = 139 \text{ ft}$$

Therefore, the ground water surface would be 2.5 ft below its level before drainage at a distance of 139 ft from the drain. Fig. 5 can be used for finding the coordinates of any point on the drawdown curve.

CONCLUSIONS

A method has been proposed for determining both the resulting flow and shape of the drawdown curve of an interceptor drain using dimensionless plots. These plots were obtained from experimental data and previously determined theoretical relationships. This method is applicable for cases in which a barrier layer is confining the flow through a relatively shallow strata and the source is either known or from engineering judgment an equivalent source is determined.

In many cases where a drain is constructed near the seepage source, such as a canal, the quantity of seepage may be increased to a large extent by the proximity of the drain.

ACKNOWLEDGMENTS

This study was performed under the technical guidance of D. F. Peterson, F. ASCE, formerly Head of Civil Engineering, Colorado State University, Fort Collins, Colo., and presently Dean of Engineering, Utah State University, Logan, Utah.

APPENDIX.—NOTATION

The following symbols adopted for use in this paper, conform essentially with "American Standard Letter Symbols for Hydraulics" (ASA Z10.2-1942), prepared by a committee of the American Standards Association with Society representation, and approved by the Association in 1942:

Dimensional characteristics are indicated in parentheses.

H = height of the original water table or head water depth above the impervious layer measured perpendicular to the bottom boundary (L). (When small slopes are being considered, heights measured perpendicular to the bottom or vertically are interchangeable);

H' = length characteristic used in Glover's formula to describe the drawdown curve (L);

h = height of the water in the tile drain above the impervious layer measured perpendicular to the slope, or tailwater depth when a tile drain is not used (L);

K = hydraulic conductivity, a constant depending on the properties of the porous medium and the water (LT^{-1});

L = upstream length of the interceptor drainage system (L);

q_b = discharge per unit width passing below the drain when the drain is placed above the impervious layer, for example, $q_t - q_d$ (L^2T^{-1});

q_d = discharge per unit width intercepted by the drain or flow into the drain, for example, $q_t - q_o$ (L^2T^{-1});

q_o = discharge per unit width from a system before the installation of an interceptor drain (L^2T^{-1});

q_t = total discharge per unit width from a system with a drain, for example, $q_d + q_b$ (L^2T^{-1});

r = radius of the tile drain (L);

S = slope of original water surface or slope of the impervious layer;

x = slope distance from the drain to a point on the drawdown curve (L). (When small slopes are being considered horizontal and slope distances are interchangeable); and

y = elevation of the drawdown curve above the impermeable layer measured perpendicular to the slope at a slope distance x from the drain (L).

DISCUSSION

M. MAASLAND,[6] **M. ASCE.**—Results on scale models for flow of water into interceptor drains are presented. The article includes checks of a formula reported by W. W. Donnan,[4] F. ASCE and extends the investigation to cases in which there is bypass flow underneath the drain. The paper is informative in that it shows that the relationships developed are valid for the range of conditions considered in the model experiments. The results are useful for certain types of drainage design. The fact that the relationships are found valid for the conditions considered by the authors does, of course, not imply that the relationships can be extrapolated to steeper slopes or to flow geometries greatly at variance with those in the experiments.

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[6] Hydr. Engr., Bur. of Reclamation, McCook, Nebr.

The writer is unfamiliar with the work of F. T. Mavis, F. ASCE and T. P. Tsui referred to[5] by the authors. However, it is obvious that the exponent $\underline{n}$ of Eq. 17, which is undefined in the paper, should equal 1 if the Dupuit differential equation is used in unmodified form. It is observed that the equation as shown in Fig. 7 is not identical to Eq. 17. The reported hydraulic conductivity of 0.038 ft per hr seems low for a sand having a mean particle size of 2.72 mm.

The shape of the water table above the drain is studied in great detail but the water table downstream of the drain is frequently of equal or greater significance. If there is bypass flow underneath the drain, particularly through highly permeable substrata below drain depth, the question arises as to whether a second or more drains will be needed further below. This leads to complex and often perplexing problems which are usually satisfactorily solved by gradually extending the drainage system as the need becomes evident. The most satisfactory analytical procedure in dealing with these problems is probably that of L. F. Ernst.[7] As is common to many groundwater problems, his methodology requires detailed knowledge of the field conditions resulting in high cost of investigation which may or may not be justified by the cost of construction of the required drainage system.

It may be noted here that an exact solution for the problem considered in the paper under discussion is given by R. Dachler[8] and by J. Kozeny.[9] This solution is obtained by a method involving the use of conformal mapping. Dachler and Kozeny discuss the validity of the formula reported by Donnan.[4] The equation given by Dachler and Kozeny can be extended to include the effect of surface-applied recharge by the simple transformation in the hodograph plane described by F. Engelund.[10] In that manner, an exact solution may be obtained for the shape of the water table, flow lines, and equipotential lines for inflow into flat drains embedded in a sloping impervious layer. This solution would be similar to the one given by Engelund,[11] for flat drains embedded in a horizontal impervious layer. An exact equation can thus be obtained for the water table which includes the effect of both a sloping impervious layer and surface-applied recharge. Another exact solution for flow into drains above a sloping impervious layer is given by Y. Gustaffson.[12]

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[7] Het berekenen van stationaire grondwaterstromingen welke in een verticaal vlak afgebeeld kunnen worden, Landbouwproefsta," by L. F. Ernst, Bodemkundig Inst. T. N. O., Groningen, Netherlands (mimeo.), No. 3, 1954, p. 55.

[8] "Grundwasserströmung," by R. Dachler, Springer, Vienna, 1936, pp. 88-98.

[9] "Hydraulik," by J. Kozeny, Springer, Vienna, Austria, 1953, pp. 411-412.

[10] "The Water Table in Equilibrium with Rainfall and Irrigation in Drainage of Agricultural Lands," by F. Engelund, Monograph 7, Amer. Soc. of Agron., Madison, Wis., 1957. pp. 128-129.

[11] "Mathematical Discussion of Drainage Problems," by F. Engelund, Transactions, Danish Academy of Tech. Science, No. 3, 1951, pp. 12-19.

[12] "Untersuchungen uber die stromungsverhaltnisse in gedräntem Boden," by Y. Gustaffson, Acta An. Suecana, 2(1), Stockholm, Sweden, 1946, p. 157.

Solution of the steady state flow problem is only a beginning and is inadequate for most practical problems. Nonstationary flow over a sloping impervious layer is considered by J. Boussinesq,[13] P. W. Werner,[14,15] F. ASCE, and others. Maasland has reviewed[16] the derivation of Werner's differential equation. These equations, which are all based on the Dupuit assumption, are only valid for flow over a moderately sloping impervious layer and are subject to further restrictions due to the assumption of approximately lateral flow which prohibits consideration of the bypass flow underneath the drain. Eqs. 12 and 13 are subject to severe restrictions for the same reasons, and caution should be exercised in applying these equations to field problems beyond the range of conditions considered in the experiments. Flow convergence near the drain cannot be overlooked and cannot be determined by the authors' methods without excessive effort.

It is, of course, clear that neither the differential equation nor the formula reported by Donnan[4] and discussed by D. K. Todd, M. ASCE, and J. Bear[17] are new or of recent data. In fact, the differential equation was first put forward and used by Boussinesq in 1877 and has been discussed in some detail by P. Forchheimer.[18] Boussinesq[19] developed the theory of the flow of groundwater further and obtained the second Glover formula reported by L. D. Dumm[20] as a special case of his general theory. In the latter work,[19] Boussinesq also considers the effect on the water table of intermittent recharge for which a more general theory was developed by Maasland.[21] The latter theory may also be used for flow over a sloping impervious layer if the Dupuit assumptions are applicable.

This discussion is not intended as a criticism of the interesting and useful work of the authors. The writer merely wishes to draw attention to the fact that there is a great deal of valuable information available in the literature which is being overlooked in present work.

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[13] "Note sur le Mouvement des Eaux Souteraines," "Essai sur la théorie des Eaux Courantes," by J. Boussinesq, Vol. 23, 1877, pp. 242-281.

[14] "On Non-Artesian Groundwater Flow," by P. W. Werner, Geofis. Pura Appl., Vol. 25, 1953, pp. 37-43.

[15] "Some Problems in Non-Artesian Groundwater Flow," by P. W. Werner, Transactions, Amer. Geophysical Union, Vol. 38, 1957, pp. 511-518.

[16] "Discussion of "Some Problems in Non-Artesian Groundwater Flow," by M. Maasland, Transactions, Amer. Geophysical Union, Vol. 39, 1958, pp. 738-740.

[17] Discussion by D. K. Todd, and J. Bear, of "Drainage of Agricultural Lands Using Interceptor Lines," by W. W. Donnan, Proceedings, ASCE, Vol. 83, No. IR3, 1959, pp. 113-115.

[18] "Hydraulik," by P. Forchheimer, Teubner, Leipzig and Berlin, Germany, 3rd Ed., 1930.

[19] "Recherches Théoriques sur l'Ecoulement des Nappes d'Eau Infiltreés dans le Sol et sur Debit des Sources," by J. Boussinesq, Journal, Math. Pures et Appl., Vol. 10, 1904, pp. 1-78.

[20] "Drain Spacing Formula," by L. D. Dumm, Agricultural Engineering, Vol. 35, 1954, pp. 726-730.

[21] "Water Table Fluctuations Induced by Intermittent Recharge," by M. Maasland, Journal of Geophysical Research, Vol. 64, 1959, pp. 549-559.

WILLIAM F. LONG,[22] **M. ASCE.**—The model study reported on should be of much interest and value to engineers confronted with problems in designing interceptor type drains. This study is an outstanding effort to gather data on the slope and configuration of draw-down curves to interception type drains. It has confirmed several theories about the slope of these draw-down curves.

There is considerable information available on the slope of draw-down curves to parallel-type relief drains; conversely, there is very little information on interception drains. This is due to the fact that a large percentage of the subsurface drains in the United States are relief-type drains and emphasis has been placed on research of relief-type drains. Interceptor-type subsurface drains are relatively new in the field of drainage. There is much research yet to be performed in this specific field. The authors and the research organization for which they work are to be commended for their efforts in this field which is relatively unknown.

In the introduction to this paper the authors state "Interception of ground water flowing laterally from a source which may be outside the affected area is the common problem." This describes a situation where all accretion to the drain is foreign to the immediate area being protected. In the writer's opinion the common situation is one where we have this accretion, from an outside source as discussed above, plus local accretion. The local accretion in the immediate area is deep percolation from precipitation, irrigation, or both, as the case may be. It is unfortunate that this model study did not include provision for evaluating the effect of this local accretion which is usually present under actual field conditions.

Information gained from evaluating a number of interception drains over a wide range of field conditions agrees with the general findings and conclusions of the authors. Draw-down curves to interception drains are relatively steep and, consequently, the affected area above interception drains is relatively small. For this reason interception drains are located near the upper edge of wet areas. The greatest effect of interception drains is usually to the area below the drain. It is hoped that future model studies will be planned to determine the position, slope, and configuration of water-table levels below interception drains. These should be planned and devised to evaluate the effect of all accretion including percolation losses from precipitation and/or irrigation.

WILLIAM W. DONNAN,[23] **F. ASCE.**—This paper is an important contribution to the literature on interceptor drains. The studies they made with the large scale model substantiate, at least in part, previously developed analytical relationships.

This writer has followed the work of Messrs. Keller and Robinson with extreme interest. Having assembled and developed some theories on interceptor drains,[4] the question arose as to how to check on the accuracy of these theories. Field investigations are often inconclusive since they are usually clouded by unknown and non-measurable variables.

After seeing a large tilting flume in the Hydraulics Laboratory at Colorado State University, Fort Collins, Colorado, the idea was formulated to use such a device as a sand tank to test the interceptor drain theories. The tilting flume could be set on various slopes and thus check on the slope variable in interception.

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[23] Agric. Engr., Western Soil and Water Mgt., Research Branch, Agric. Research Service, U. S. Dept. of Agric., Pomona, Calif.

The existing flume at Fort Collins was not available for use so a new 70-ft flume was built. It may be of interest to note that at the conclusion of the interceptor drain research, a wall was knocked out of the Hydraulics Laboratory and the two tilting flumes were joined together to create one of the longest indoor tilting flumes in the world for use in hydraulic research work.

In a model as large as this 70-ft long tilting flume, the problem of getting uniformity of the porous media and uniformity of compaction of the material presents difficulties. Uniform compaction was derived by using a pneumatic vibrator of the type used in pouring concrete.

It seems apparent that the success of the Keller and Robinson project with this large model should give impetus to more research on drainage problems with large tilting tanks or flumes. More work needs to be carried out on flow into drains and on optimum location of the interceptor drain with respect to the reservoir, canal or other source of seepage. In addition, careful studies should be made of the flow phenomenon adjacent to and immediately down slope from the interceptor drain.

HERMAN BOUWER.[24]—The authors report on a sand-model study of steady, two-dimensional flow systems in a porous medium. These flow systems, however, are also suitable for analysis by analog such as resistance networks or others. It would be interesting to compare the results in this paper with data obtained by analog to determine, for instance, whether solution by analog might not be a more feasible approach for studying problems of the type discussed in the paper. A resistance network analog would be especially adapted in this case, since it affords simultaneous solution of the flow above as well as below the water table and water-table shapes are found as products of the analysis.[25]

The authors worked with a rather course sand with a "capillary rise" of 1.5 in. This may justify their assumption of no bypass flow above the drains. In the light of presently available data, however, the order of magnitude of the height of capillary fringes above water tables in field soils may be 1 ft, less for coarse soils, more for structureless clay soils. For shallow flow systems such as in this study, therefore, the capillary fringe may be of sufficient height to render the flow above the water table of considerable importance. Water may pass over the drains as tension flow in the capillary fringe and continue its seepage downhill. To also intercept the flow above the water table, it would be necessary to use an open ditch as interceptor, or, if tile drains are desired to backfill the trench with a coarse material or to line it with an impermeable membrane.

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[24] Senior Agric. Engr., Southwest Water Conservation Lab., Western Soil and Water Mgt. Research Branch, Soil and Water Conservation Research Div., Agric. Research Service, U. S. Dept. of Agric., Tempe, Ariz.

[25] "A Unifying Numerical Solution for Two-Dimensional Steady Flow Systems in Porous Media with an Electrical Resistance Network," by Herman Bouwer and W. C. Little, Soil Sci. Soc. Amer., Vol. 23, 1959, pp. 91-96.

JAN VAN SCHILFGAARDE.[26]—The authors present some excellent data on the flow of water through gravel towards interceptor drains together with parts of a theoretical solution. They show that theory and experimental data are in close agreement. However, they fail to make full use of this agreement when they do not show that the theory they present is fully adequate to solve the problems that are posed, both as to flux and shape. Eq. 10 apparently is a solution of the differential equation

$$q_t = K y \left(\frac{dy}{dx} + S\right) \ldots\ldots\ldots\ldots\ldots (19)$$

with the substitution $H = q_t/KS$. Here H is not used with the same meaning as elsewhere in the paper. This H is the thickness of the aquifer where $dy/dx = 0$, that is, where $x = \infty$, and not the height of the aquifer at $x = L$. The authors resolve this dilemma by introducing $H' = q_t/KS$ in Eq. 11, so that H may be reserved for the head water depth. It should be noted that the "adjustment" of Eq. 10 by the introduction of H', as proposed by the authors, implies a misinterpretation of the origin of Eq. 10. Eq. 11 results directly from a solution of the writer's Eq. 1 when one integrates between the limits 0 to x on x and h to y on y.

Since $H' = q_t/KS$, the value of H', or of the total flow through the aquifer, q_t, may be determined from Eq. 11 by the substitution $x = L$ and $y = H$. Rearranging into dimensionless groups, this substitution yields

$$\frac{H}{LS} - \frac{h}{LS} + 1 = \frac{q_t}{KLS^2} \ln \frac{\dfrac{q_t}{KLS^2} - \dfrac{h}{LS}}{\dfrac{q_t}{KLS^2} - \dfrac{H}{LS}} \ldots\ldots\ldots (20)$$

Thus, it is possible to calculate, by means of the theory, the total flux from the parameters of the problem, just as Fig. 3 enables such determination from the experimental data. Fig. 8 eliminates the need for solving Eq. 20 by trial and error and provides a graphical solution which parallels the authors' Fig. 3.

In case $S = 0$, a solution of the same differential equation results in the authors' Eq. 17 with the exponent $n = 1$. An intermediate result is

$$\frac{q_d}{K} = \frac{q_t}{K} = \frac{\left(H^2 - h^2\right)}{2L} \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Fig. 9 is a copy of the authors' Fig. 6 with a plot of Eq. 21 superimposed on it. Thus the agreement between theory and experiment, claimed by the authors for the shape analysis, can again be extended to the flux analysis. To illustrate, the authors' first example gives $H/LS = 5$ and $h/LS = 2$; Fig. 8 yields $q_t/KLS^2 = 14.5$. Hence, with $K = 0.0001$ fps, $q_t = 0.000029$ cfs; $q_b = KhS = 0.000004$ cfs; $q_d = q_t - q_b = 0.000025$ cfs. This compares with $q_d = 0.000026$ cfs as

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[26] Assoc. Prof. of Agric. Engrg., N. C. State College; and Drainage Engr., Soil and Water Conservation Research Div., Agric. Research Service, U.S.D.A., Raleigh, N. C.

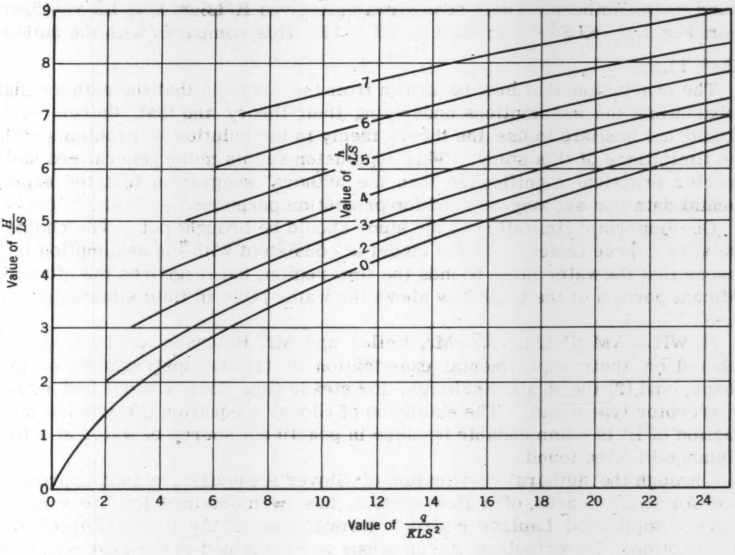

FIG. 8.—GRAPHICAL SOLUTION OF EQ. 20

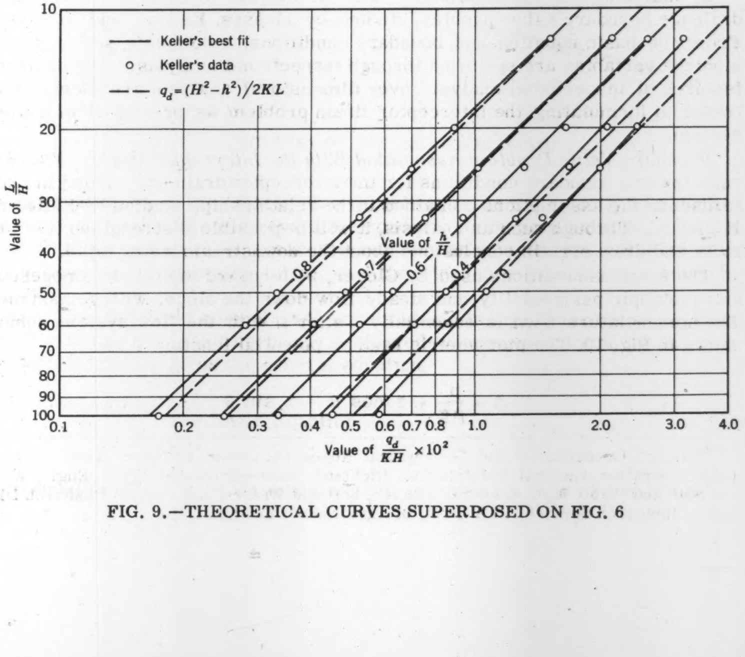

FIG. 9.—THEORETICAL CURVES SUPERPOSED ON FIG. 6

found by the authors. Their second example gives $H/LS = 1$, $h/LS = 0.5$, and, from Fig. 1, $q_t/KLS^2 = 1.2$; thus $q_t/KS = 12$. This compares with the authors' $H' = 11.9$.

The conclusion that may be drawn from the above is that the authors' data substantiate the assumptions underlying their theory and that, therefore, one should not hestiate to use the theory freely in the solution of problems within the limitations of this study. This conclusion seems more generalized and of greater practical significance than the authors' suggestion that the experimental data per se, may be used for prediction purposes.

One important limitation of the study should be brought out. The choice of the very coarse material in the model is consistent with the assumption in the theory that the water table bounds the flow region, but it ignores the often significant portion of the total flow above the water table in field situations.

R. WILLIAM NELSON.[27]—Mr. Keller and Mr. Robinson are to be complimented on their experimental examination of (1) the upstream water table shape, and (2) the drain discharge, for steady flow down a slope and into an interceptor type drain. The extension of Glover's equation through the introduction of H' is commendable because in practice a source of water at a finite distance is often found.

Through the authors' verification of Glover's equation, a good approximation for some phases of a flow system, has been obtained for the exact but more complicated Laplace equation through use of the Dupuit-Forchheimer Assumptions. Nevertheless, assumptions are contained in the expression, and inaccurate predictions may result in some phases of the flow problem. Careful study of the desirable and limiting features of Glover's equation will provide the judgment which is so necessary for realistic use of an approximation.

In analyzing the assumptions it will be helpful to state the interceptor drainage boundary value problem studied by Messrs. Keller and Robinson. From the basic equation and boundary conditions, a complete set of dimensionless variables are specified through inspectional analysis. The desirable features of inspectional analysis over dimensional analysis are nicely illustrated in formulating the interceptor drain problem as presented in the next section.

Boundary Value Problem Associated With the Interceptor Drain.—The basic equation and boundary conditions for the interceptor drain are needed in order to discuss the assumptions contained in the relationships studied by Keller and Robinson. Through such an analysis, it will be possible to strengthen their results and draw certain conclusions about the downstream water table.

The same assumptions used by Glover, with regard to soil of homogeneous and isotropic permeability and steady flow down the slope, will be assumed. The nomenclature used is essentially Keller's with the flow system schema shown in Fig. 10. The piezometric head or potential function Φ is

$$\Phi = \frac{p}{\rho g} + z \cos \theta - x \sin \theta \qquad (22)$$

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[27] Engr., Geochemical and Geophysical Research, Chem. Effluents Tech., Hanford Labs. Operation, General Electric Col, Richland, Wash.; formerly Agric. Engr., Western Soil and Water Mgt. Research Branch, Soil and Water Conservation Research Div., Agric. Research Service, U. S. Dept. of Agric., Fort Collins, Colo.

From Darcy's Law the velocity components are

$$v_x = -K\frac{\partial \Phi}{\partial x} \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

and

$$v_z = -K\frac{\partial \Phi}{\partial z} \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which $\frac{p}{\rho g}$ is the pressure head, Φ denotes the piezometric head or potential function which is a function of x and z, ρ is the density, g refers to the gravitational scalar, v_x is the velocity in x direction, v_z is the velocity in z direction, K denotes the saturated hydraulic conductivity or permeability, y is the distance to the water table above the impermeable layer, the value of y is only defined along the water table), and the other symbols are as shown in Fig. 10.

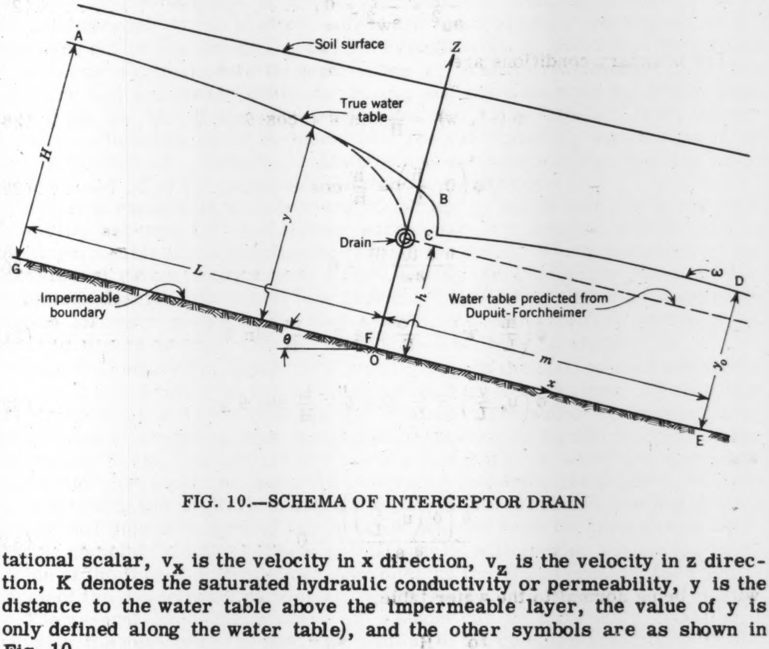
A
Soil surface
Z
True water table
H
y
B
Drain
C
G
L
Impermeable boundary
θ
F
O
h
Water table predicted from Dupuit-Forchheimer
ω
D
m
x
y_0
E

FIG. 10.—SCHEMA OF INTERCEPTOR DRAIN

Since flow in planes normal to the drain shown in Fig. 10 are identical, then only flow in the x-z plane need be considered whereupon combining Darcy's Law with continuity gives the general equation.

$$\text{The Basic Equation, } \frac{\partial^2 \Phi}{\partial X^2} + \frac{\partial^2 \Phi}{\partial Z^2} = 0 \ldots\ldots\ldots\ldots (25)$$

is the equation which must be satisfied by Φ in order for it to be a solution.

The boundary value problem can be expressed more efficiently in dimensionless variables and parameters. Using the boundary parameters shown in Fig. 10 as scaling factors, let $u = \frac{x}{L}$, $w = \frac{z}{L}$, and $\phi = \frac{\Phi}{H}$. Substituting these values into Eq. 22 results in the dimensionless potential ϕ being,

$$\phi = \frac{p/\rho g}{H} + \frac{L}{H} w \cos \theta - \frac{L}{H} u \sin \theta \quad \ldots\ldots\ldots\ldots (26)$$

Eq. 25 reduces to dimensionless form upon changing the variables in accordance with the previous listing

$$\frac{\partial^2 \phi}{\partial u^2} + \frac{\partial^2 \phi}{\partial w^2} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

The boundary conditions are

$$\phi\,(-1,\ w) = \frac{L}{H} \sin \theta + \cos \theta \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

$$\phi\left(0,\ \frac{h}{L}\right) = \frac{h}{H} \cos \theta \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

$$\frac{\partial\ \phi\ (u,\ 0)}{\partial w} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

$$\phi\left(\frac{m}{L},\ w\right) = \frac{y_o}{H} \cos \theta - \frac{m}{H} \sin \theta \quad \ldots\ldots\ldots\ldots (31)$$

$$\phi\left(u,\ \frac{y}{L}\right) = \frac{y}{H} \cos \theta - \frac{L}{H} \sin \theta\ u \quad \ldots\ldots\ldots\ldots (32)$$

and

$$\frac{\partial \left[\phi \left(u,\ \frac{y}{L}\right)\right]}{\partial\ n} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

with n being normal to the water table.

$$\frac{y_o}{L} = \frac{H}{L} - \frac{Q_d}{L\ K \sin \theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

The only simplifying boundary assumption made is in Eq. 29 in which the drain has been considered as a point sink rather than a drain of finite radius r. This assumption is justified as pointed out by Keller and Robinson.

A set of dimensionless parameters which completely describe the flow system (ϕ as a function, F) are immediately obtained from inspection of Eqs. 27 through 34, that is

$$\phi\ (u,\ w) = F \left[\frac{L}{H},\ \theta,\ \frac{h}{H},\ \frac{y_o}{H},\ \frac{m}{H},\ \frac{y}{H}\right] \ldots\ldots\ldots\ldots (35)$$

Through the inspectional procedure we are assured that the resulting set of dimensionless ratios (Eq. 35) completely describes the system.

Upon comparison of the dimensionless variables in Eq. 35 with those of Keller, m/H and y_o/H are not in the latter's analysis. Yet in the experimental procedure the investigators had to arbitrarily set these ratios with y_o/H being made equal to h/H at some m/H. With y_o/H being arbitrarily set at values of m/H, the usual check on completeness in the dimensional analysis could not function. In other words, the consistency of data when plotted in terms of the dimensionless variable does not assure in this case completeness with respect to these two variables since the experiment did not allow y_o/H to vary. Accordingly there may be problems in the experimental system used from having held the two ratios essentially constant.

The flume experiment of Keller and Robinson could be thought of as an analog computer which in effect solved the basic equation meeting all of the boundary conditions except Eq. 34. This condition on y_o could have been found by successive adjustments through setting y_o to some reasonable value, then measure Q_d, the steady drain discharge, and compute what y_o should have been from Eq. 34. If the calculated value of y_o was different from the set value, readjustment could be made until the calculated y_o was the same as that in the tail box. Once the calculated and set values agree then the basic equation and all of the boundary conditions are satisfied.

Keller's experimental procedure of setting y_o equal to h for some m/H implicitly assumes the downstream water table is a straight line parallel to the impermeable layer and passing through the drain. The significance of the unexamined ratios really reduce to how closely the Dupuit-Forchheimer Assumptions describe the actual flow system. If they are a rather good approximation for some phases of the flow system, then the arbitrary setting of y_o equal to h may not have affected those phases of Keller's study.

Dupuit Compared to Laplacian Formulation.—In the last section the rigorous Laplacian formulation for the interceptor drain was obtained. To this time the problem has defied exact analytical solution. (The author has pursued such an analytical solution in considerable detail through conformal mapping of the derivative of the complex potential onto a half plane, on which the Hodograph plane was previously mapped. An integration involving the angle θ required for a Schwarz-Christoffel transformation has to date prevented solution by such a method.) However, several things can be learned from the formulation without the complete solution. These partial results will provide a basis for evaluation of the Dupuit-Forchheimer Assumptions thereby suggesting the significance of the downstream parameters which were not considered.

The general downstream water table shape can be rigorously established by using the results of the previous section in the complex velocity or Hodograph plane. Brooks presented[28] the results establishing the downstream water table shape in his discussion of Donnan's paper,[4] so detailed discussion here is unnecessary. Briefly it was found, with Brooks qualitatively substantiating this data with a sand tank model, that: 1) Immediately downstream of the drain there is a stagnation point (point C in Fig. 10). 2) The water table never intersects the drain. 3) The downstream water table gradually rises with increasing x, i.e., between C and D the water table has a positive slope

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[28] Discussion of "Drainage of Agricultural Lands Using Interceptor Drains," by R. H. Brooks, Proceedings, ASCE, Vol. 85, No. IR4, December, 1959.

(dy/dx > 0). Accordingly, the actual downstream water table shape for the interceptor drain is as shown schematically in Fig. 10. For comparison, the dashed lines representing the predicted water table shape based upon the Dupuit-Forchheimer Assumptions are shown in Fig. 10.

In the experimental work of Keller, y_0 was always set smaller than it should have been. Accordingly, the upstream water table would be lowered slightly near the drain with an ever-decreasing amount at greater distances upslope from the drain. Perhaps more significantly a smaller y_0 would cause the drain to intercept less of the water than might otherwise have been removed. The change in drain outflow would be expected to be a large error in comparison with the upstream water table difference which probably is quite insignificant.

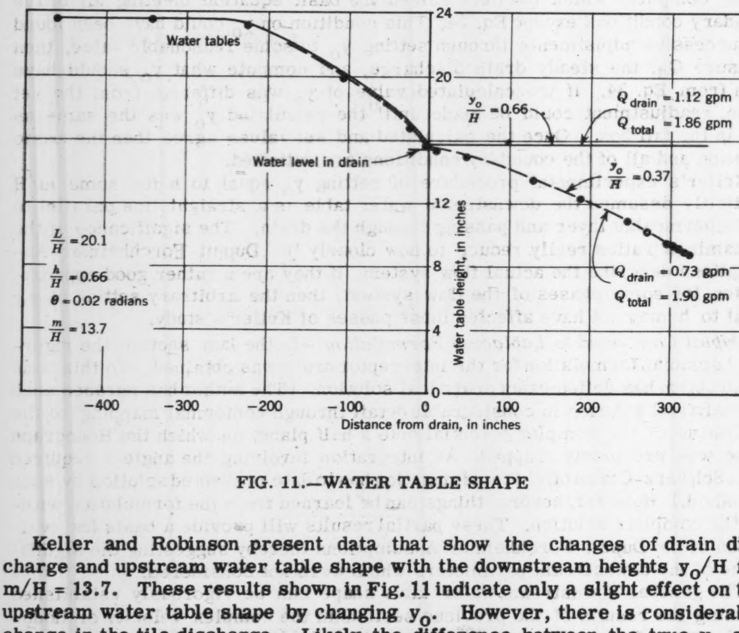

FIG. 11.—WATER TABLE SHAPE

Keller and Robinson present data that show the changes of drain discharge and upstream water table shape with the downstream heights y_0/H for $m/H = 13.7$. These results shown in Fig. 11 indicate only a slight effect on the upstream water table shape by changing y_0. However, there is considerable change in the tile discharge. Likely the difference between the true y_0 and that used (h) was usually a third or fourth of the change shown in Fig. 11. Unfortunately, the true difference isn't known so no definite conclusion can be made regarding the probable bias in the observed drain discharges.

From the above discussion and results shown in Fig. 11, the Keller-Robinson conclusion—that the upstream water table is adequately described by the Glover equation—is very much justified. The same certainty is not justified with regard to the predicted tile discharge although it may be correct. Certainly it is the best available estimate of discharge and probably is not in error by more than 15%.

Computational Aids for Glover's Equation.—Experience with Glover's equation in a design method[29] for hetergeneous soils indicated an easy method of calculating the drawdown curve would be helpful. Accordingly, Mr. Donald Miles devised a series of curves, Fig. 12, incorporating the results of Keller and Robinson, which allows rapid graphical solution of Glover's equation. The value of H' can also be found through use of the progressive graphs.

More description of the examples given in Fig. 12 may be helpful. The known conditions are: H = 11.5 ft, h = 6 ft, L = 2,000 ft, and s = 0.002 ft per ft. The value for H'-h must be found in order to get the drawdown curve. This is accomplished in the example in Fig. 12 by entering at 2,000 on the x scale (L and H are just a limiting pair of x and y values) and following the dashed lines to get H'-h/H. This gives H'-h as 15 for later use in the upper right hand example.

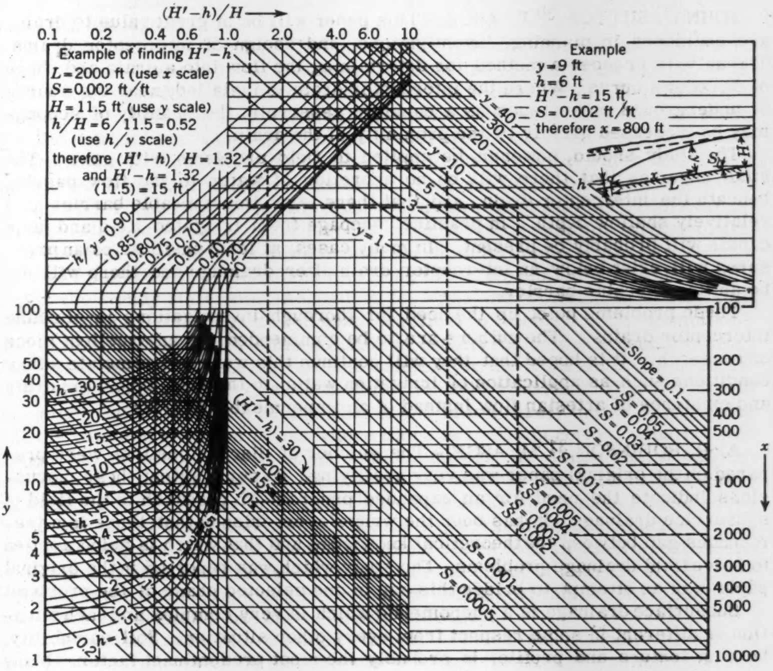

FIG. 12

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[29] "A Design Method for Interceptor Drains in Saturated Heterogeneous Soils," by R. William Nelson. Submitted for publication in Agric. Engr.

To get the complete drawdown curve it is easiest to assume a value of y and calculate the associated x. Assuming y = 9 (see right hand example, Fig. 12) and entering the left hand y scale move horizontally to h = 6, hence upward and following the curved dashed line, meanwhile continuing horizontally along y = 9 to (H'-h) = 15, then upward to the intersection with the earlier curved dashed line, thence horizontally to y = 9, then down to s = 0.002 and horizontally thereby finding x = 800 ft. By successively assuming values for y and finding the x in a like manner, the entire drawdown curve is easily and quickly found.

JOHN G. SUTTON,[30] F. ASCE.—This paper will be of great value to drainage engineers in planning, investigations, and design of interceptor drains. The authors propose a method for determining the flow into a drain and shape of drawdown curve. Where the interceptor drain is installed, near the source of underground water, such as a canal or reservoir, the quantity of seepage may be increased greatly by the proximity of the drain.

The study should, however, be applied to field problems with care. The study assumes that there will be no accretion of water to the flow passing beneath the interceptor drain, and that there is an impermeable barrier at a relatively shallow depth. In practice, seepage from irrigated field and farm canals will probably be present. In many cases, at least small artesian pressures will exist below an interceptor drain after drainage and there will not be an impermeable barrier.

These problems point up the need for thorough investigations in planning interceptor drains. The authors are to be commended for an excellent piece of research. It is hoped that they will continue this work and simulate other conditions such as application of irrigation water to the area below the drain and existence of artesian flow instead of an impermeable barrier.

A. R. ROBINSON,[31] M. ASCE.—The number of discussions that were prepared on the original paper was very gratifying. To some extent these discussions indicate the need for an expanded program of research in the field of subsurface drainage. It has been the writer's experience that there is a tremendous gap between the theoretical knowledge and that which is being applied to solve field drainage problems. The study which was reported in the original paper was an attempt to bridge this gap in one selected phase of the problem.

Subsurface drainage is a phenomenon which is very complex and each situation is different in some respect from every other situation. Soil variability, both in texture and profile, is probably the most predominant factor. Many variations in hydraulic conductivity for soils of similar texture but with different structural characteristics are common. The hydraulic conductivity also may vary with moisture content. Flow in the zone of partial saturation is very complex and has not received adequate treatment. Many studies, both laboratory and field, have been made of flow below the water table, that is fully saturated. In order to reduce the number of variables, in many cases laboratory studies have been idealized to the point of questionable adaptation. The results from many field evaluations are inconclusive because numerous unmeasured variables were not considered.

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[31] Agric. Engr., Western Soil and Water Mgt. Research Branch, U. S. Dept. of Agric., Agric. Research Service, Soil and Water Conservation Research Div., and Colorado Agric. Experiment Sta., Fort Collins, Colo.

The discussions have pointed out several areas in which this model study was limited or deficient. Messrs. van Schilfgaarde and Bouwer indicated the importance of flow above the water table. They point out that the amount of flow in this region of partial saturation may be appreciable. In this study the material was very coarse so that the so-called capillary fringe was approximately 1.5 in. in height. The amount of flow in this zone was undoubtedly insignificant. It is certainly true that flow in the zone of partial saturation is important in drainage considerations and has received little attention from a research standpoint.

Maasland, Sutton, Donnan, Nelson, and Long each point out the importance of the shape of water table downstream from the drain. This portion of the water table was arbitrarily fixed in the study that was reported. Nelson pointed out the relationship of the downstream condition to the upstream one. He shows that the downstream water table in the model study was always set to a smaller depth than it should have been. As a result, the upstream water table would be lowered from that which was observed. This condition would also cause the drain to intercept less of the flow than would have otherwise been removed under the natural condition. Possibly the most pressing need for further drainage research is in the area downslope from the interceptor drain.

It was pointed out by Maasland and Long that the problem has been simplified to the steady state one. According to Maasland, solution of steady state flow problems is inadequate for most practical problems. It is the author's observation that the steady state solution is usually the only one used for practical problems. As a general rule, the solution for the transient state becomes so involved that it is rarely ever used for the field situation. This is unfortunate since the steady state assumption is one which rarely exists. Long states that the common situation is one where there is local accretion, usually from irrigation, as well as flow from an outside source. He states that it was unfortunate that the effect of this local accretion was not evaluated in the reported study. It is certainly true that this effect should have been evaluated although the model study was not intended as a general study of drainage but was to include only one selected phase. Maasland states that an exact equation can be obtained for the water table which includes the effect of both the sloping impervious layer and surface applied recharge. Maasland implies that this solution is not yet available but can be easily obtained.

Mr. Maasland seemed to be highly disturbed that a considerable amount of information, from which he quotes, was not used for background material for the study. He also states that neither the basic differential equation nor the formula reported by Donnan (1) and used in this paper as Eq. 10 were new or of recent date. It is recognized by those trained in the field of flow in porous media that the developments of Boussinesq and Forchheimer were made. The so-called Glover formula (Eq. 10) was derived independently by Glover from heat flow analogies and is being used for interceptor drain design. It should be re-emphasized that the reported study was not meant to be a general thesis on the entire field of interceptor drainage but was only intended to encompass certain portions of the problem. Mr. Maasland was correct in pointing out that the reported hydraulic conductivity of the material seemed very low. This conductivity should have been given as 0.038 ft per sec.

Mr. Donnan points out some of the developments that led to initiation of the study. The recognition of need for the study as well as some of the preliminary planning was made by Mr. Donnan. He points out that there should be more emphasis on drainage research conducted in large tilting flumes. The author agrees that there are many phases which can be studied in a large tank. The large model gives a scaled physical picture of the problem which makes the information obtained more understandable. However, as pointed out by Bouwer, an electrical analog is especially adapted for studies of this type. Data can be collected, using an analog, at much less expense and in a shorter time. The accuracy of the data should be as good or better than when using the large equipment. As stated by Mr. Bouwer, the resistance network analog affords a simultaneous solution of flow above, as well as below, the water table.

Two significant developments by van Schilfgaarde and Nelson are of note: Mr. Nelson's starts with the basic equation and gives the boundary conditions. From these equations he selects the dimensionless parameters that completely describe the flow system. This he terms inspectional analysis in contrast to dimensional analysis, which was used in the original paper. Using this procedure, the geometry of the downstream water table was included in the problem. Mr. van Schilfgaarde rearranged the original Eq. 10 into dimensionless form, which yielded his Eq. 20. From this, he obtained Fig. 8 which is a more usable solution than that obtained from Fig. 3 in the original paper.

From the comments of Mr. van Schilfgaarde relative to the introduction of H', an explanation is needed. In the original derivation of Eq. 10 it was evidently not recognized that in certain cases the flow in the system would not be constant before and after drain installation. In the case of a system where there is an increase in flow, a term was needed that would include the original depth, H, plus some additional depth to compensate for the additional energy in the system. The sum of these two was given the term, H', so as to not confuse the original depth of flow H.

The computational aid for the solution of Eq. 11, which was prepared by Mr. Nelson is very commendable. This will allow rapid solution of the equation for either shape of the drawdown curve or flux.

Mr. van Schilfgaarde stated that the author's data substantiate the assumptions underlying their theory and that therefore one should not hesitate to use the theory freely for solution of problems within the limitations of this study. This statement is certainly true and the major limitations should be repeated. The conditions were; (1) a sloping, impermeable boundary existed at some measurable distance below the water table, (2) a defined source existed at some determined distance from the drain location, and (3) a source that was constant in elevation and able to supply additional flow as needed to satisfy the system.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 3260

WORLD PRACTICES IN WATER MEASUREMENT AT TURNOUTS

By Charles W. Thomas,[1] F. ASCE

With Discussion by Messrs. Lee Chow; and Charles W. Thomas

SYNOPSIS

Measuring devices at farm turnouts on open channel irrigation systems are discussed under six general functional classifications. Illustrations of each classification in use in different parts of the world are cited. Conclusions regarding the type of device best suited to meet local requirements are not drawn. However, by directing attention to the different techniques followed, a design incorporating the desirable elements of several of these techniques might eventually be developed.

INTRODUCTION

Successful operation of an irrigation system depends on adequate control and measurement of flows. At no place is this more important than at the farm turnout. It is at this point that the individual user and the operators meet. On any irrigation system, the largest number of structures of any one type are the farm turnouts. There are some 85,000 on lands served by systems built by the United States Bureau of Reclamation, Dept. of the Interior, and another 75,000 are to be found in irrigated areas served under the Warren Act and special lands. Outlets on canals in the Punjab region of India and Pakistan numbered over 41,000 before 1950 and more have been added since that time. Such facts should indicate the importance of these structures and the reason for drawing attention to the means of controlling and measuring flows through them.

Note.—Published essentially as printed here, in June, 1960, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2530. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

[1] Hydr. Engr., Bur. of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

There are many considerations to be taken into account in the selection of a control and measuring device to best fit conditions at the site. Loss in head, economy of installation and maintenance, range of discharge to be handled, legal requirements, are among the factors that make the selection difficult.

DESIGN AND OPERATION OF IRRIGATION SYSTEMS

General.—A review of some of the general aspects of irrigation systems is helpful to appreciate fully and understand the problems associated with control and measurement of water at the farm turnout.

Good design of an irrigation system is based, primarily, on making maximum use of the water resources available. In general, the land area that may be irrigated is established by the amount of water resource. By proper regulation and control of the water, the area served may be extended.

In the design of the system, the quantity of water to be conveyed is based on the acreage to be served. A study of rainfall, soil types, crops that may be grown, depth to the water table, and other factors determine this quantity.

Method of Distribution of the Water.— To make the best use of the water in the irrigation system, the designer must consider its distribution. One of the following general methods may be used:

1. Continuous flow deliveries
2. Rotation or intermittent flow
3. Demand deliveries
4. A combination of these methods.

In an irrigation system designed for continuous flow, each irrigator is supplied his share of water in a continuous stream. This results in large land owners receiving large flows and small owners small flows. To measure the delivery at each turnout, devices in the system would necessarily vary in size.

The rotation or intermittent flow system is based on delivering a fixed amount of water to each farm at definite intervals. Storage reservoirs are necessary to make the water available and permit delivery at definite intervals during the irrigating season. Systems, deriving their source of water from normal stream flow, must make deliveries at the time the water is available. A more standardized size of measuring device can be used under this system and, often, a saving in the number of measuring devices could result.

The demand-delivery system is designed to make water available at all the farm turnouts on call or demand by the farmer. It is certainly the most convenient and economical from the consumers' standpoint. If adequate storage and ample conveyance capacity are provided, and if there is sufficient control of flows to avoid wastage en route or off the end of the system, then the deliveries can be made on a call or request by the farmer. This system of delivery is not adaptable to projects operating without storage facilities. For such projects the water must be used when available and cannot be supplied to meet the demands of individuals. Measuring devices can be standardized, to some degree, under this system. Control is of major importance.

Some irrigation projects operate with a combination of these systems. During periods of high seasonal runoff, the main conveyances are usually operated to capacity if the demand exists and if there are legal rights to the water. At such times, at least a major portion of the farmers desiring water can be served. When storage reserves are being drawn upon, the project may operate on rotation or demand.

Other combinations of means of distribution may be used. Combinations of distribution methods cause added problems in control and measurement.

Charges for Water.—The charges made for water supplied from the irrigation system vary considerably throughout the world. There are, however, four general charging categories into which the majority of systems will fall.

The first system is a charge based on the rate of flow. This necessitates a rate of flow measurement and adequate records. The second, a volume basis, necessitates a volumetric measuring device or a rate-of-flow device combined with a time record of deliveries. The third method is a charge based on the acreage of crops matured. The rates vary in accordance with the crops grown. Such items as the amount of water necessary to produce a certain crop, the season when most water is necessary for the crop (winter or summer), the value of the crop and comparative cost of irrigation from wells or other means, all enter the formula from which the charge is derived. Measurements of flow as a basis for charges are not necessary under this system, but adequate control must be exercised to insure equitable distribution. The fourth method is a charge based on each irrigation for a given area. In climates that do not demand continuous irrigation for the successful growth of crops or in locations in which the same crop is grown in large areas year after year, a fixed charge is possible. This system is practiced extensively in rice-producing areas.

Operating Organization.—In many areas of the world, there is, generally speaking, a large organization to control the conveyance and delivery of the water, to regulate the flows, and to keep records necessary for charges.

In other parts of the world, especially in India and Pakistan, there is generally no manual control on the farm turnouts. The design is such that they work automatically and continuously. In the Punjab area, the lateral systems, with a maximum capacity of from 300 to 400 sec-ft, are so designed that manual control or regulation is not necessary at any point in the system except at the turnout from the main canal. The internal distribution of water among the various cultivators on the laterals is generally managed by the cultivators themselves, and the government, which operates the canals, keeps no records of the farm deliveries.

METHODS OF MEASUREMENT AND CONTROL

From the foregoing presentation, it may be seen that the operation of an irrigation system could be simplified and many of the problems of controlling and measuring the water delivered to the consumers could be solved if some reliable means were available to deliver automatically a fixed, predetermined discharge to each consumer under the system. Technicians have been striving for many years to devise such a device. Many of the schemes offered have never outgrown the paper stage. Others have proven worthless in actual practice or have been discarded because of their complexity and unreliability. There are, however, some methods of treating the problem that have met with success.

The Six General Schemes.—Knowledge acquired by the author during extensive travel assignments, and through studies made on advanced research fellowship in France suggests that measuring devices at the farm turnout may be classified broadly according to their operation. For the purpose of this discussion, these measuring devices are divided into six broad functional classifications. No clear lines can be drawn between these selected classifications, and some examples given overlap into other categories.

The six general classifications are: (1) those devices, sometimes referred to as modules, that will automatically deliver a constant, or near constant, discharge over a range of changes of both upstream and downstream water levels; (2) those devices, sometimes referred to as semimodules, that will automatically deliver a constant, or near constant, discharge over a range of downstream levels but in which the discharge will vary with changes in the upstream water level; (3) those devices that provide an equitable distribution of flow over a range of fluctuations of upstream water surface levels (these devices are generally semimodules); (4) those devices designed to deliver, automatically, a constant discharge when operated in conjunction with auxiliary equipment to control the upstream water level and that are not affected, within limits, by changes in downstream levels. (5) the structures and equipment, in general use in the United States, that give a range of discharges depending on upstream and downstream heads and which may require auxiliary means of manual control; and (6) those devices that do not serve as controls, but which totalize the discharge, over a relatively wide range of flows passing them, and, thus, provide an equitable basis for charges.

It is not the intent of the author to attempt an explanation of the exact design and operation of the various devices mentioned or to argue the many advantages or possible weak points of each. The classification has been made on a functional basis and it is assumed that if the device, structure, or combination installation is designed, constructed, and operated as intended, the function will be fulfilled.

The examples cited constitute only a minor portion of all those developed or suggested. There are many others throughout the world and, possibly, many of those not mentioned are more popular.

Modules.—As previously stated, a device that would automatically deliver the desired quantity of flow to the farm regardless of fluctuations of water surface in the conveyance system or in the farm ditch would very nearly solve the problem of control and measurement at the farm turnout.

Many of the old irrigation networks of the world operate on direct flow from the streams without benefit of seasonal storage. Many of these systems are quite extensive. Such operation causes considerable fluctuation in water levels in the conveyances. In periods of high seasonal runoff, the canals may be operated to capacity, but as the stream flow subsides, the quantities available for diversion become limited and the canal levels fall. In these areas, much work has been done toward evolving a turnout to meet these conditions.

S. I. Mahbub and N. D. Gulhati have traced the history[2] of evolution of various types of turnouts on the canals of the Punjab in India and Pakistan. Operation of some of the large canal systems in this area began shortly after 1800. The authors define the module as a device that will deliver automatically a fixed quantity of water regardless of fluctuations of levels of water surface, within limits, in the canal or in the delivery. They cite a number of such devices in use. Some are constructed with moving parts and others without.

One such device cited, that has no moving parts, is known as the Gibbs Module, Fig. 1. The water is lead through an inlet pipe to a spiral rectangular trough that is level and covered. Free vortex flow develops in this eddy chamber with a consequent rise in water surface at the outer wall. A number of cross baffles, with bottom edges sloped, correctly serve to skim off part of the

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[2] "Irrigation Outlets," by S. I. Mahbub and N. D. Gulhati (revised and enlarged by N. D. Gulhati), Atma Ram & Sons, Kashmere Gate, Delhi, India, 1951.

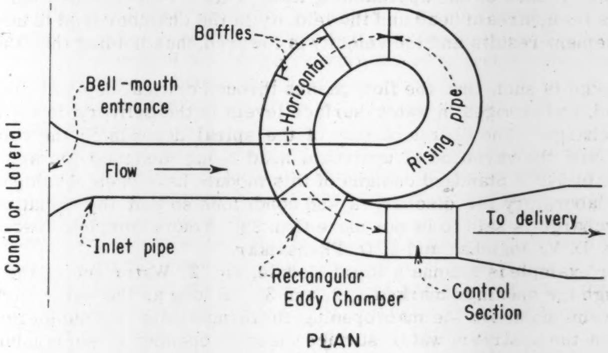
Baffles
Bell-mouth entrance
Canal or Lateral
Flow
Horizontal
Rising pipe
To delivery
Inlet pipe
Rectangular Eddy Chamber
Control Section
PLAN

FIG. 1.—GIBB'S MODULE

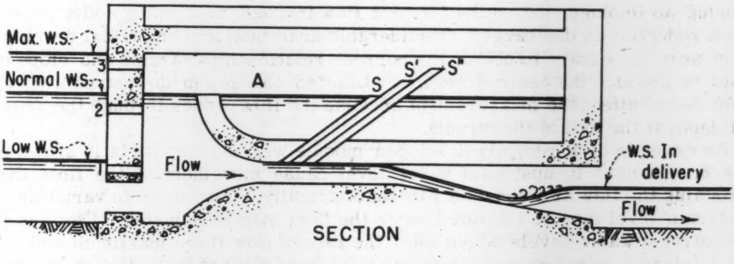
Max. W.S.
3
Normal W.S.
2
Low W.S.
1
A
S
S'
S"
Flow
W.S. In delivery
Flow
SECTION

FIG. 2.—KHANNA'S RIGID MODULE

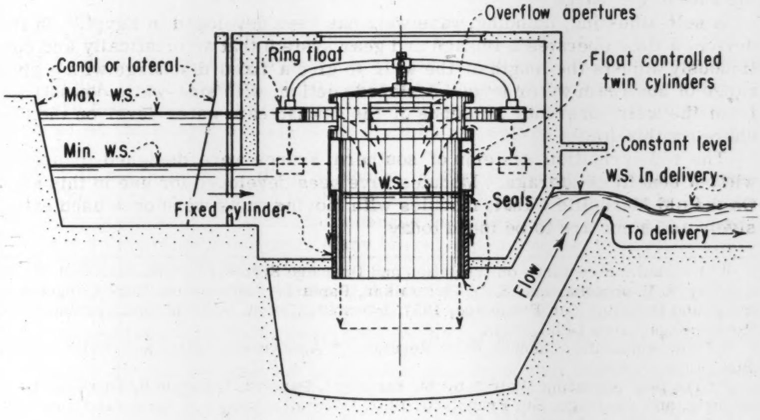
Overflow apertures
Ring float
Canal or lateral
Float controlled twin cylinders
Max. W.S.
Min. W.S.
W.S.
Constant level
W.S. In delivery
Seals
Fixed cylinder
To delivery
Flow

FIG. 3.—MODULE WITH MOVING PARTS

flow and turn it back on the approaching flow. This action becomes more pronounced as the upstream head and the velocity in the chamber tend to increase. An impingement results and the velocity is lowered, thus holding the discharge constant.

The design is such that the flow passes through critical depth at the downstream end, and changes in water surface levels in the delivery do not reflect in the discharge. The degree of turn of the spiral depends on the volume of discharge and the variation of upstream head being designed for and varies from 180°, to 540°. Standard designs of this module have been developed in the hydraulic laboratory for discharges and conditions so that the variation from design discharge is said to be not more than 3%. A more complete description[3] is given by D. V. Joglekar and S. D. Phansalkar.

Another example is Khanna's Rigid Module, Fig. 2. Water enters the structure through the openings marked 1, 2, and 3. As long as the water surface in the canal remains above the main opening, the turnout acts as a submerged orifice. When the upstream water surface rises to opening 1, some eddies are formed at the entrance and the flow is reduced below that which would normally be discharged with the increased head. Additional upstream head causes chamber A to fill, and water flows successively into the inclined slots, S, S', and S", causing an impingement on the forward flow through the opening and a consequent reduction in discharge. Considerable analytical and empirical development were necessary to obtain the correct relationships of size and slope of slots to produce the desired results. Limited changes in downstream water level do not affect the measurement because the flow passes through the critical depth at the exit of the turnout.

An example of a recently developed module with moving parts is shown in Fig. 3. Changes in upstream water level cause movement of the float that maintains the flow at a constant rate automatically. Considerable variation in upstream level may be tolerated since the float may be adjusted. Changes in downstream water levels do not alter the rate of flow if the maximum level in the delivery does not exceed the level of the fixed weir at the outlet of the distributor. This exit weir, developed in Italy, has been calibrated for measuring the rate of discharge.[4]

A self-adjusting, standing, wave weir has been developed in Egypt.[5] In this device, a float operates a linkage and gear system that automatically and continuously adjusts the length of the weir to give a fixed discharge with a given range of upstream water level. A sloped section, with side-walls downstream from the weir, prevents influence of the downstream water level on the discharge within limits.

The old irrigation systems of southern France were designed to operate without benefit of storage. Modules have been developed for use in this area. On the old Marseille Canal, modules with moving parts were once used extensively and some are to be found today.

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[3] "Hydraulic Structures on Irrigation and Drainage Systems for Measurement of Water," by D. V. Joglekar and S. D. Phansalkar, Paper 19, Question 9, Third Congress on Irrig. and Drainage, San Francisco, 1957, Internatl. Comm. on Irrig. and Drainage, 104, Sunder Nagar, New Delhi, India.

[4] "New Automatic Channel Flow Regulator," Anonymous, Water and Water Engrg., June, 1956, p. 250.

[5] "The Self Adjusting Weir," by M. Elmadani, Paper 2, Question 9, Third Congress on Irrig. and Drainage, San Francisco, 1957, Internatl. Comm. on Irrig. and Drainage, 104, Sunder Nagar, New Delhi, India.

Semimodules.—A semimodule is a device that will pass a fixed discharge, provided either the head water or tail water remains constant. Most of such devices are designed in such a manner that changes in tail water conditions do not reflect in the rate of discharge. Variations in upstream water surface elevations will, however, change the discharge rate. Such a device is, theoretically, better than one that is sensitive to variations in both upstream and downstream water levels.

Starting with a turnout consisting of a cut, and later, a barrel passing the canal bank. Both were subject to changes in discharge with changes in both upstream and downstream levels, the next step would normally be to envolve a device of the semimodule type.

In many parts of the world, the water user's interference with the turnout led to placing checks, cisterns, orifices, and similar structures at the downstream ends of the turnouts to raise the water surface to a level that would be above the maximum level in the delivery channel. Thus, the turnout would not be affected by operational changes made by the water users.

In the Punjab, the Harvey-Stoddard Improved Irrigation Outlet, Fig. 4, was developed as one such type of device. A fixed crest weir maintains the level at the end of the turnout above the maximum water level in the farm delivery. In the same area, the open flume type of turnout was modified by placing a roof block in the flume, Fig. 5, to cause the flow to pass through critical depth and thus not reflect changes, within limits, in downstream levels. Further description and details of semimodules may be found in a paper[6] by Hamid.

Weirs placed in the conveyance downstream from the turnout, as used in the United States, are not included in this classification. Although the weir does not prevent changes in the downstream levels from affecting the flow through the turnout, provided the weir is not submerged, the weir structure is nearly always separated from the turnout structure and the control remains at the turnout gate.

Equitable Distribution of Flow.—This type of measuring device is so designed that the levels in the conveyance system may vary over a wide range, and all deliveries will remain proportional. This insures an equitable distribution without providing a control device at each individual turnout for periodic regulation.

In areas where charges are based on acreage of crops matured or where water is distributed in the quantity available on the basis of land areas irrigated or for other reasons, an equitable distribution of the available water to the users may suffice. In the Punjab, for instance, measurement is made at the head of the lateral and equitable distribution serves as a basis for farm deliveries.

Open flume turnouts of the type shown in Fig. 6 serve this condition quite well. The entrance is shaped so that an equitable share of the flow is extracted. Exit conditions are such that changes in downstream level are not reflected to an appreciable degree. Many barrel-type turnouts, to serve a similar purpose, have been developed.

Divisors are used in many parts of the world to effect equitable distribution of flow. These structures divide the flow into the desired proportions. To be effective, upstream and downstream flow conditions must be similar across the

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[6] "Distribution and Measurement of Irrigation Supplies in West Pakistan," by Chowdhry Abdul Hamid, Paper 18, Question 9, Third Congress on Irrig. and Drainage, San Francisco, 1957, Internatl. Comm. on Irrig. and Drainage, 104, Sunder Nagar, New Delhi, India.

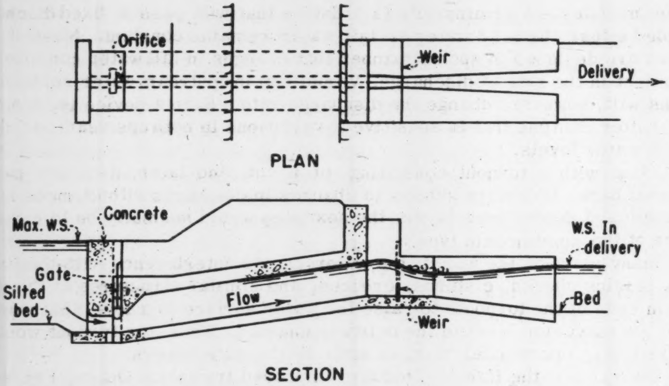
Orifice
Weir
Delivery
PLAN
Concrete
Max. W.S.
W.S. In delivery
Gate
Silted bed
Flow
Bed
Weir
SECTION

FIG. 4.—THE HARVEY-STODDARD IMPROVED IRRIGATION OUTLET

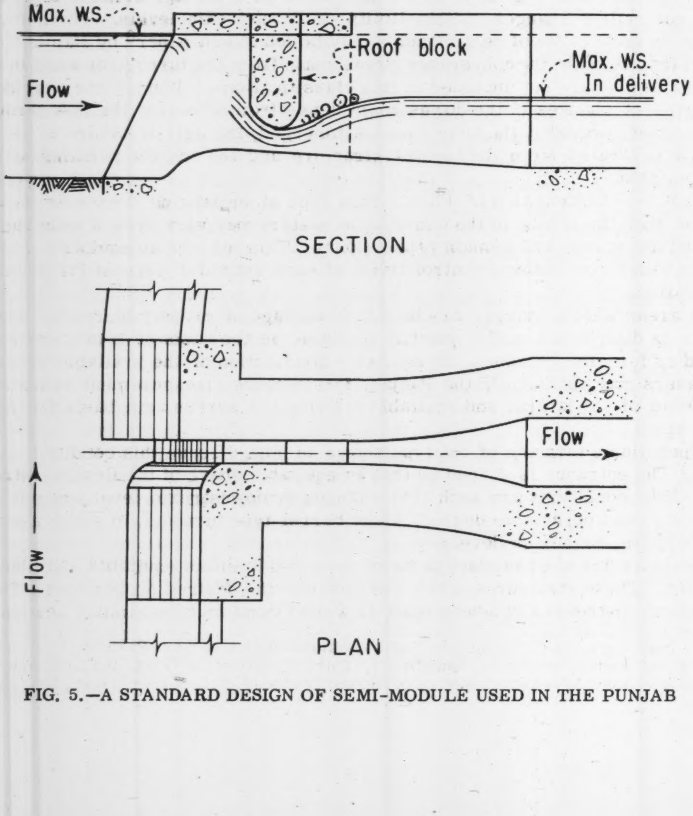
Max. W.S.
Roof block
Max. W.S. In delivery
Flow
SECTION
Flow
Flow
PLAN

FIG. 5.—A STANDARD DESIGN OF SEMI-MODULE USED IN THE PUNJAB

section. The dimensions of the openings are not necessarily in the same proportions as the desired division of discharge. They may be made rigid or variable. Fig. 7 shows a divisor in an old irrigation system near Marrakech, Morocco. This structure also serves the purpose of slowing the flow, that is above critical velocity, for diversion into the turnout.

In other irrigation networks in southern Europe and North Africa, a divisor developed in France is used (Fig. 8). The dividing blade is adjustable and calibrations are made so that the flow may be proportioned between the two channels in varying amounts. The standard setting for channels on fairly flat gradients is illustrated in this figure. When steep slopes are encountered, the setting is slightly different.[7]

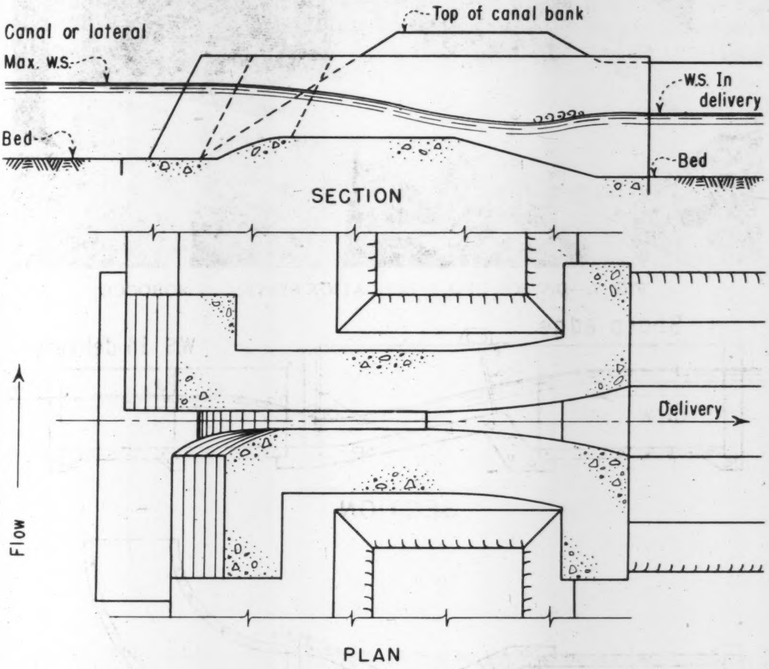

FIG. 6.—CRUMP'S OPEN FLUME OUTLET

In many of the old irrigation systems in the United States, "division boxes" were used. There were generally constructed of wood. Many of them were of fixed proportions. One type of adjustable divisor is shown in Fig. 9. This particular structure, as well as others of similar proportions, was calibrated for flow measurement at different settings over a range of upstream heads.[8]

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[7] "Quelques Considerations sur l'Equipment des Reseaux d'Irrigation NEYRPIC," Grenoble, France.

[8] "Divisors (for the measurement of irrigation water)," by V. M. Cone, Bulletin 228 of the Agric. Experiment Sta. of the Colo. Agric. Coll., Fort Collins, Colo., April, 1917.

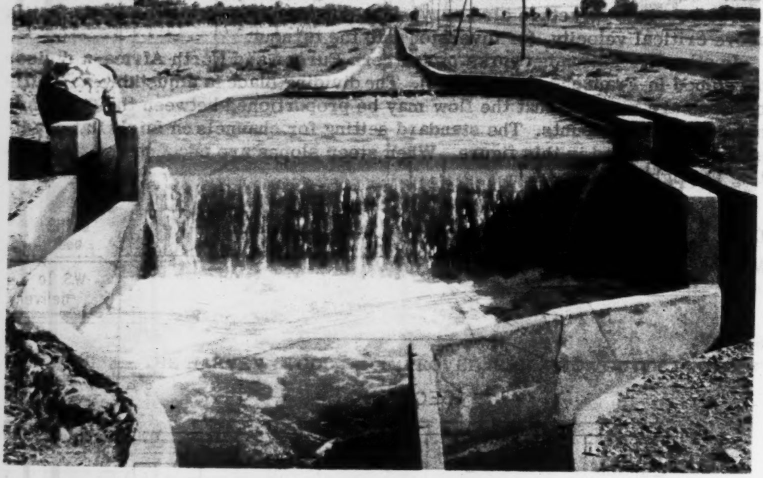

FIG. 7.—DIVISOR IN OLD IRRIGATION SYSTEM IN MOROCCO

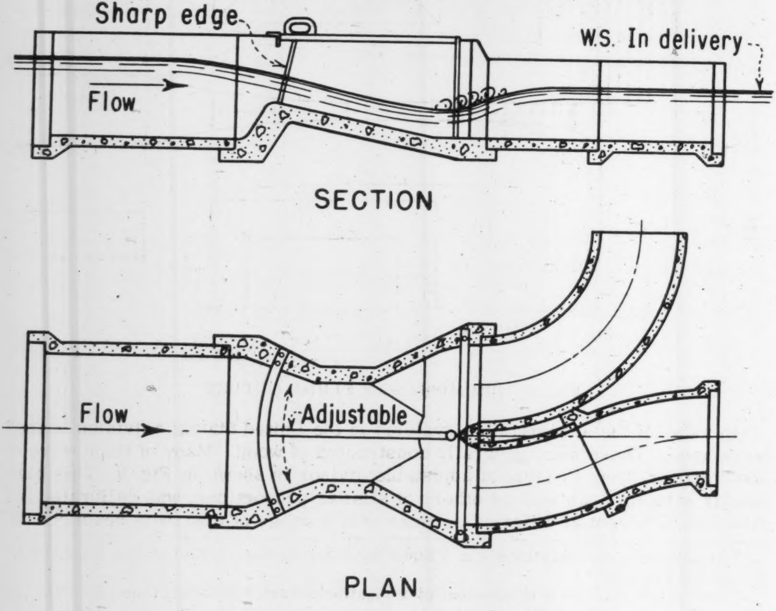
Sharp edge
W.S. In delivery
Flow
SECTION
Flow
Adjustable
PLAN

FIG. 8.—FRENCH TYPE PROPORTIONAL DIVISOR

Semimodules With Auxiliary Apparatus.—The design of a turnout that will deliver a fixed quantity of water and not be subject to variations of upstream and downstream water levels may become very complicated as the examples cited would indicate. Such devices may also be subject to operational difficulties. The effects of changes in downstream water levels can be removed by relatively simple design. It would seem then that the use of auxiliary equipment to control the level upstream would effect a workable solution.

Examples of distributors that will deliver a near constant quantity will first be explained, followed by examples of auxiliary means employed to control, automatically, the water surface upstream from the distributor to a near constant level, and negate the need for periodic regulation of each turnout.

The distributor shown in Fig. 10 is used extensively in southern Europe and North Africa and, to a lesser extent, in other parts of the world. The sliding plates are either fully open or fully closed. The desired discharge is obtained by opening one or more of the passages. It may be seen from the cross section in Fig. 11 that each section of the distributor is formed by a specially shaped sill and a fixed plate. The sill and fixed plate are contained between vertical parallel side plates, thus creating an orifice.

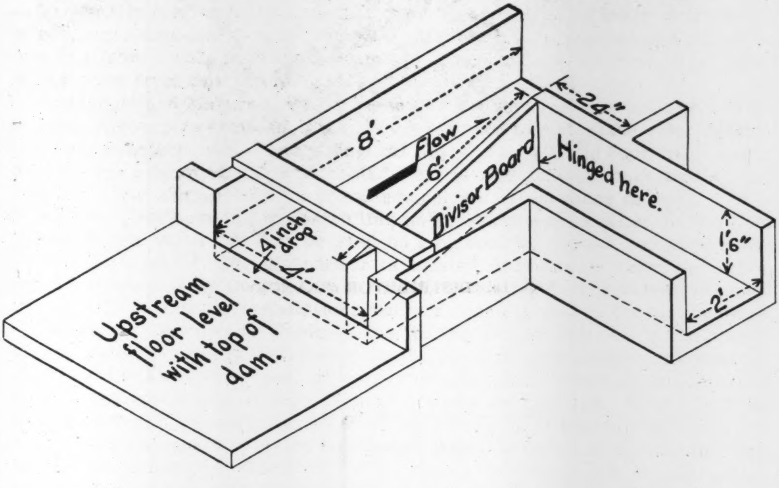

FIG. 9.—U. S. TYPE PROPORTIONAL DIVISOR

If the water level above the orifice is controlled to permit variation between predetermined limits, the discharge that passes through the orifice remains nearly constant. The stability of the discharge is maintained because the increase in velocity, that results from the increase in head, is accompanied by a greater degree of contraction of the outgoing stream. A discharge curve is included in Fig. 11.

The downstream slope of the sill causes the formation of a hydraulic jump in which part of the kinetic energy is recovered. Thus, the overall loss in head through the apparatus is low. The formation of the hydraulic jump also makes

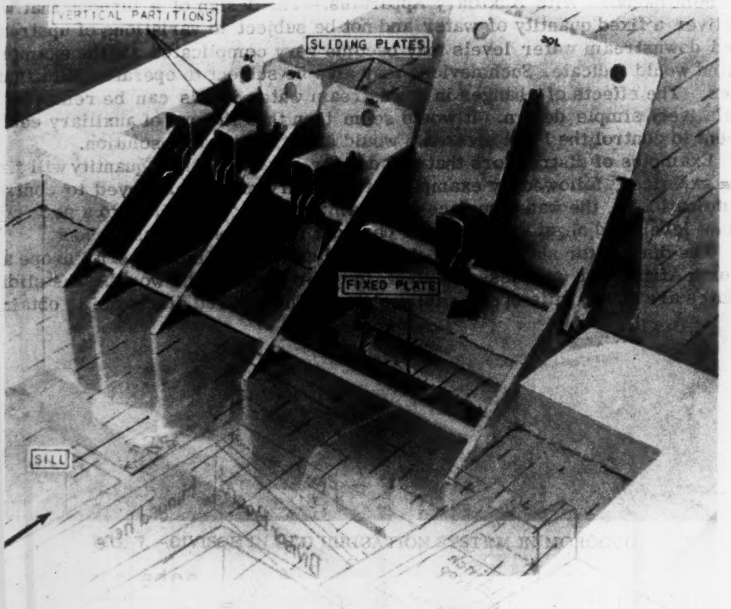
VERTICAL PARTITIONS
SLIDING PLATES
FIXED PLATE
SILL

FIG. 10. DISTRIBUTOR (FRANCE)

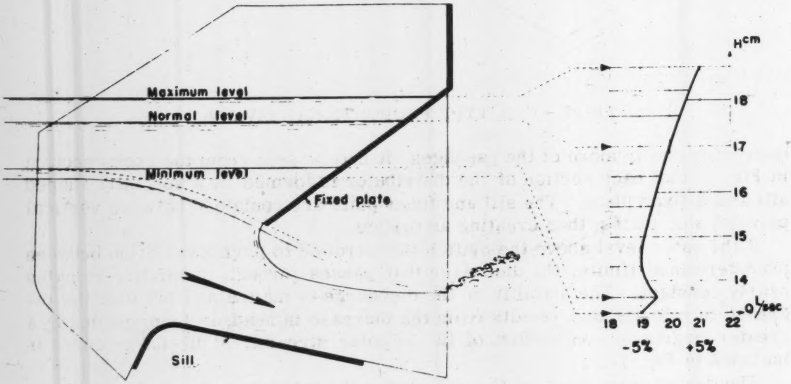
Maximum level
Normal level
Minimum level
Fixed plate
Sill
H cm
18
17
16
15
14
Q l/sec
18
19
20
21
22
-5%
+5%

FIG. 11.—DISCHARGE CURVE OF DISTRIBUTOR

the discharge independent of the downstream water level as long as the jump is not drowned.

Another distributor designed along very similar lines is shown in Fig. 12. The distributor can absorb a greater change in upstream level because of the action of the siphon and the consequent added impingement on the jet. A discharge curve is also shown in the figure. Although developed in France and in North Africa, these distributors have been studied in other parts of the world.[9]

A number of schemes have been devised for controlling automatically the water surface in a canal or lateral. They may be used in conjunction with distributors. A balanced radial gate with a float attached to the upstream skin plate, Fig. 13, automatically provides a nearly constant water surface upstream from the gate. This automatic control gate is installed in the canal or lateral just downstream from the turnout equipped with a distributor.

A balanced gate for automatically controlling the level downstream from the installation has also been developed (Fig. 14). The float, in the foreground, automatically positions the gate leaf over an orifice to maintain a predetermined, near-constant level. This gate is used on storage reservoirs or on turnouts from canals in which the water level varies. The distributor is placed downstream from this gate (Fig. 15).

In Algeria and Morocco, where the newer conveyances consist, generally, of precast concrete sections, other means are used to control the water surface to a near constant level. Long, diagonal weirs, Fig. 16, and "duckbill" weirs, Fig. 17, are employed. The "duckbill" weir is also used in conventional lined sections. The distributors are installed upstream from these devices.

Another development for controlling water surface automatically is a float operated, disc valve that may be used on a turnout fed from a source that has a variable level.[10] The distributor is located downstream from the valve. Fig. 18 shows a large installation to illustrate this type. The valves are usually applied to smaller installations, but work equally well in the larger sizes.

Measuring Devices in Common Use in the United States.—In the United States, water is usually distributed on the basis of rate-of-flow, in most instances cubic feet per second, although this rate, combined with time, may be used to give a volume, in acre-feet, as a basis for charges. Weirs, critical depth structures, Parshall flumes, and similar means of measurement require that the head be known to obtain discharge. When the water surface in the conveyance is permitted to fluctuate, adjustment of a turnout gate or similar installation is necessary to obtain the desired head at the point of measurement. The usual practice is to make this adjustment daily and then obtain a reading of head on the measuring structure to serve as a basis of charges and to control distribution.

As a general rule, only the larger metering stations are equipped with apparatus to record the head continuously at the measuring section. Farm turnouts are thus excluded. An exception to this practice may be found in areas in which the cost of water is high. Because the discharge varies with the head and the head is subject to variations with changes of water level in the canal or lateral, assuming the operator is not present at all times to adjust the turnout gate, it is probable that the rate-of-flow does not remain entirely constant

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[9] "Automatic Regulator to Obtain Constant Discharge in a Channel for a Widely Varying Income Flow," by Mushtaq Ahmed, Mian Muzaffar Ahmed, and S. A. Awan, Paper 5, Question 9, Third Congress on Irrig. and Drainage, San Francisco, 1957, Internatl. Comm. on Irrig. and Drainage, 104, Sunder Nagar, New Delhi, India.

[10] "Amenagement de la Basse Moulouya (Maroc oriental)," by Florent Granger, Terres Et Eaux, No. 26.

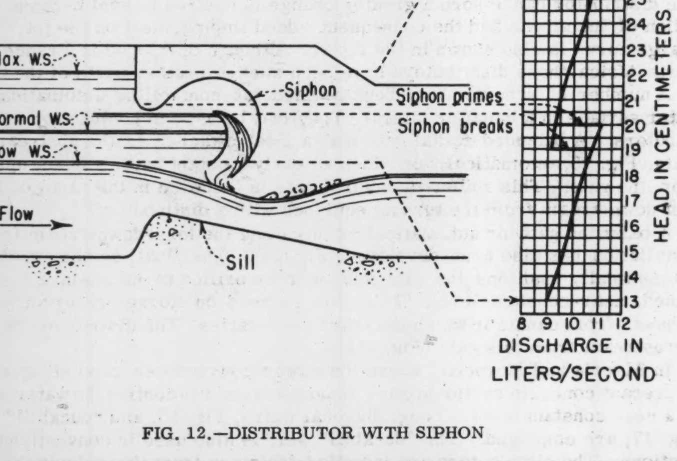
Max. W.S.
Normal W.S.
Low W.S.
Flow
Sill
Siphon
Siphon primes
Siphon breaks
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
HEAD IN CENTIMETERS
8
9
10
11
12
DISCHARGE IN LITERS/SECOND

FIG. 12.—DISTRIBUTOR WITH SIPHON

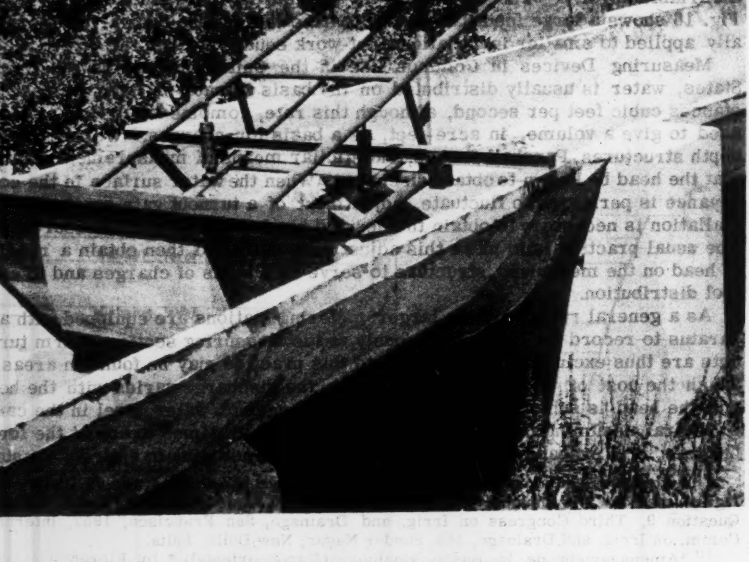

FIG. 13.—CONSTANT UPSTREAM LEVEL AUTOMATIC GATE

during the interim between observations of the gage height. Because major changes in flow are not made over short periods of time, the rate of discharge at the turnout remains relatively constant. However, a continuous record of gage height would be desirable. The present cost of providing such a record at each farm turnout is not justifiable in most systems.

Generally speaking, the measuring devices employed at farm turnouts on open channel systems are designed to permit variation of the upstream head and, in most cases, variation of the downstream head. Observation of the upstream head or the differential head and the use of tables or curves prepared from previous calibrations permit evaluation of the discharge. Regulation of the head is, almost without exception, by manual control.

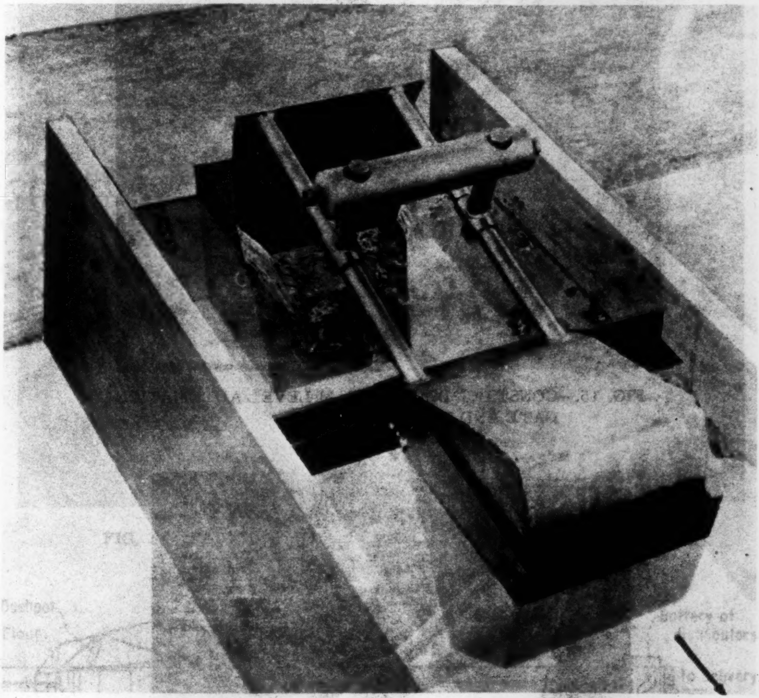

FIG. 14.—CONSTANT DOWNSTREAM LEVEL AUTOMATIC GATE

There are installations in which the water levels in canals and laterals are controlled automatically, but this automatic control has not reached to the farm turnout. The controls are usually employed to raise the water surface a sufficient amount to permit deliveries and not necessarily to provide a constant head on the delivery. Examples of automatic control are to be found, but these are exceptions.

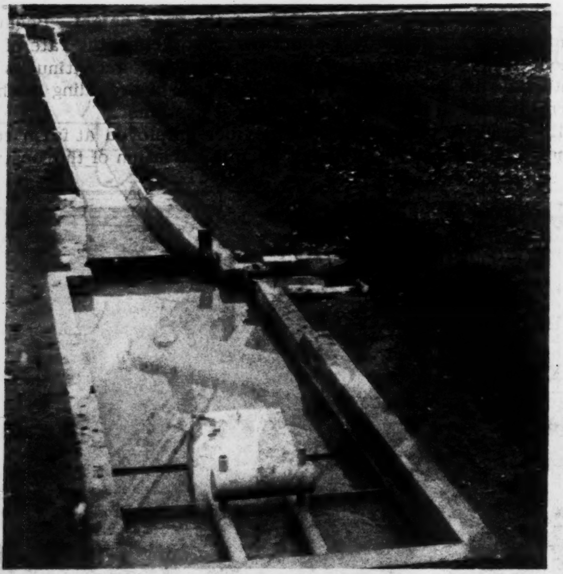

FIG. 15.—CONSTANT DOWNSTREAM LEVEL AUTOMATIC GATE AND DISTRIBUTOR

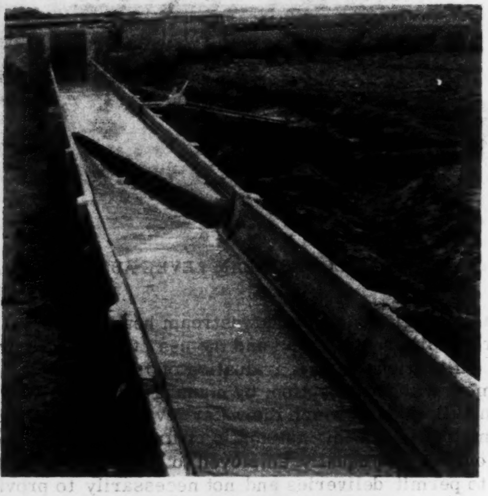

FIG. 16.—DIAGONAL WEIR IN PRECAST CONCRETE LATERAL

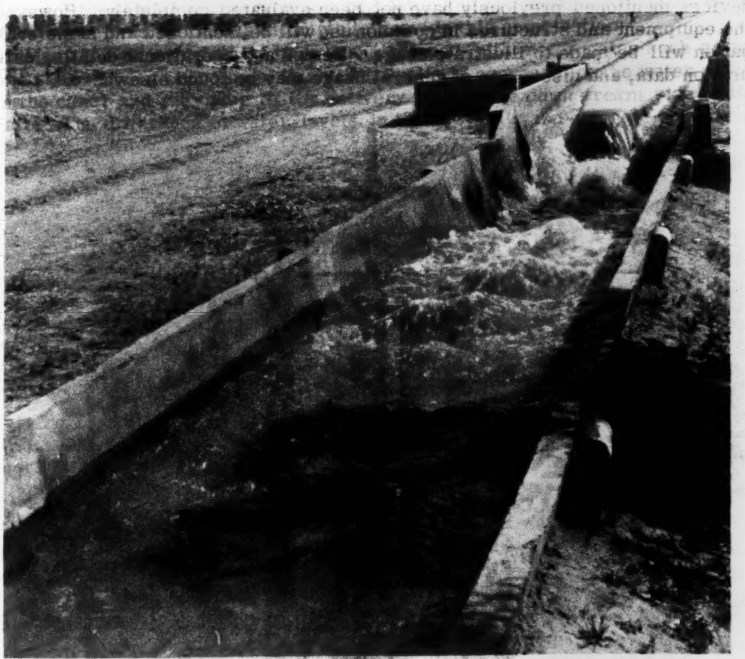

FIG. 17.—DUCKBILL WEIR IN PRECAST CONCRETE LATERAL

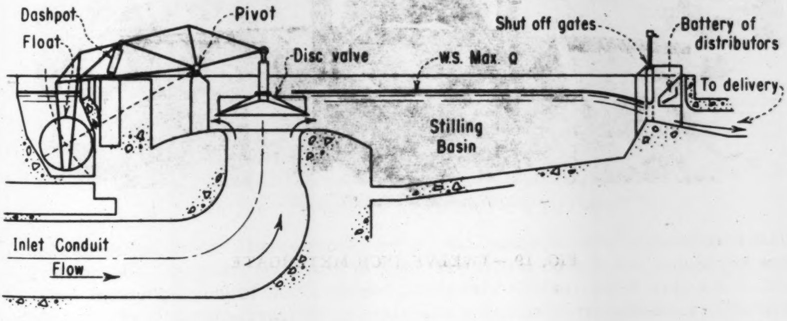
Dashpot
Float
Pivot
Disc valve
W.S. Max. Q
Shut off gates
Battery of distributors
To delivery
Stilling Basin
Inlet Conduit
Flow

FIG. 18.—DISC VALVE CONSTANT LEVEL REGULATOR

It would not be consistent to evaluate the measuring devices in use in the United States with regard to their operational characteristics, because those devices mentioned previously have not been evaluated completely. However, the equipment and structures in common use will be mentioned and some evaluation will be made to illustrate usage. Design and operational details, calibration data, and discussion of general usage may be found elsewhere.[11, 12]

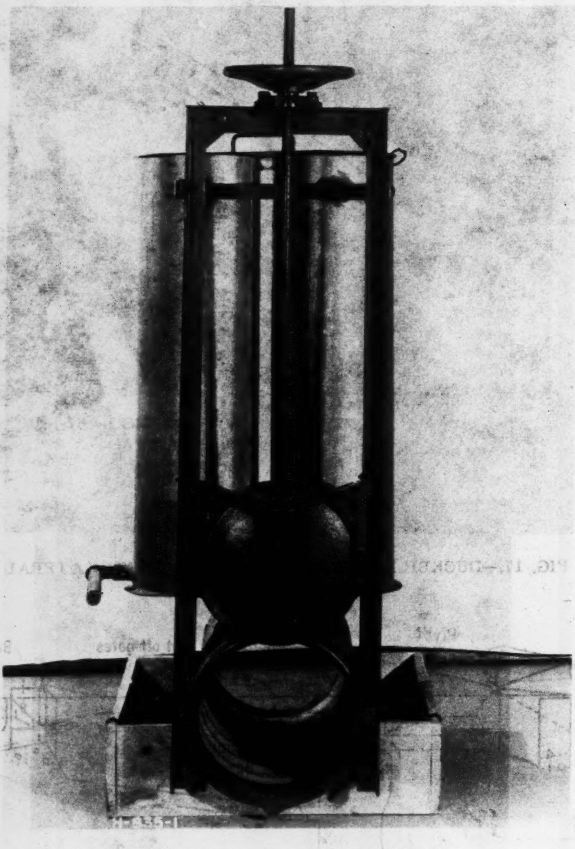

FIG. 19.—TWELVE-INCH METERGATE

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[11] "Water Measurement Manual," Bur. of Reclamation, U. S. Dept. of the Interior, Denver, Colo., May, 1953.

[12] "Structures and Methods for Measuring Irrigation Water," by Charles W. Thomas, Paper 9, Question 9, Third Congress on Irrig. and Drainage, San Francisco, 1957, Internatl. Comm. on Irrig. and Drainage, 104, Sunder Nagar, New Delhi, India.

A shutoff is, in most instances, required at a farm delivery because of the manner in which the systems are operated. Economy in installation can be effected if the shutoff also serves as a regulator and as a means of measurement. There have been numerous attempts to provide this combination.

Many irrigation systems have been equipped with meter gates such as the one shown in Fig. 19. One of the stilling wells is connected to the water prism in the canal and the other to the delivery pipe on the downstream side of the gate. The difference in water levels in the two wells and the gate opening is measured and the discharge obtained from tables developed from standard calibrations.[13] The total head loss is low. The initial cost is low compared with other means because of the combined features of measurement, regulation, and shutoff. Periodic observations and manual adjustments are necessary. Changes in either upstream or downstream water levels alter the rate of flow. Other

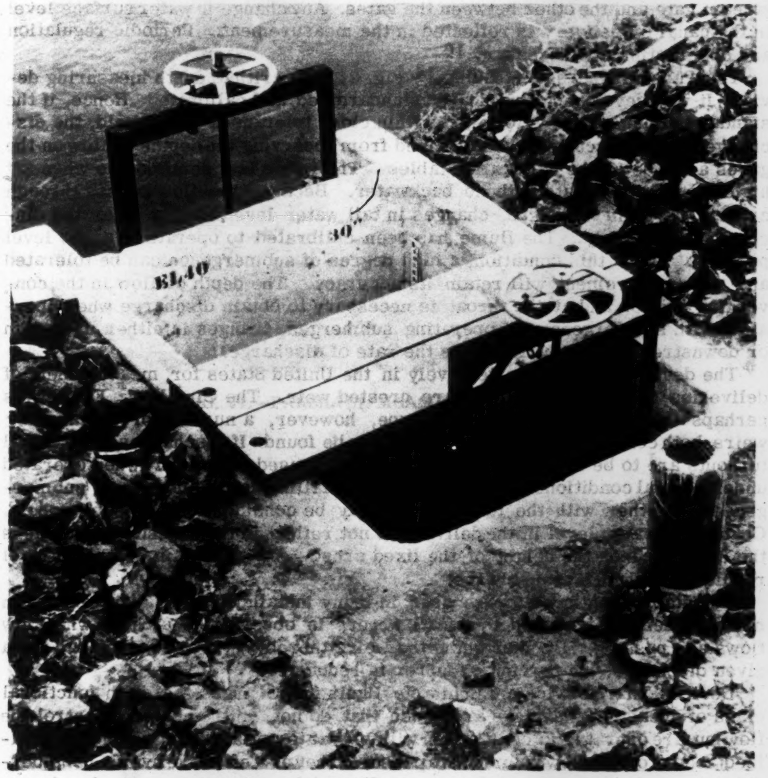

FIG. 20.—CONSTANT HEAD ORIFICE TURNOUT

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[13] "Water Measurement Tables for the Armco Metergate," Armco Drainage and Metal Products, Denver, Colo., 1951.

types of gates have been calibrated, but the tables are not as complete and the gates are not as well standardized.

The constant head orifice turnout shown in Fig. 20 has become popular for measuring farm turnouts as well as installation in canals and laterals. The operation is manual. Calibration and standardization have been accomplished in the smaller sizes. The upstream gate is set at the required opening, as shown in the rating tables, to deliver the desired quantity. The downstream gate is regulated until there is a 0.2-ft differential head across the upstream gate as indicated by two enameled scales, one located upstream from the upstream gate and the other between the gates. Any change in water surface level in the canal or delivery is reflected in the measurement. Periodic regulation and observation are necessary.[10]

The Parshall measuring flume,[14] Fig. 21, is a critical depth measuring device. It was developed empirically, standardized, and calibrated. Hence, if the standard dimensions are followed within close tolerances in the field, the discharge is quite accurate when obtained from observing the depths of flow on the gages and use of the calibration tables. The flume is a semimodule when set high enough to operate with no backwater. Because the flow passes through critical depth in the flume, changes in tail water levels do not alter the discharge appreciably. The flume has been calibrated to operate in a low level position. Under this condition, a high degree of submergence can be tolerated and the measurement will retain its accuracy. The depth of flow in the converging section and at the throat is necessary to obtain discharge when operating with submergence. If operating submerged, changes in either upstream or downstream levels will change the rate of discharge.

The device used most extensively in the United States for measurement of deliveries to the farm is the sharp crested weir. The Cipolletti, Fig. 22, is perhaps the most frequently used type, however, a number of rectangular weirs, both contracted and suppressed, may be found. If discharges from small turnouts are to be measured, a V-notch weir is used. The weir, when operated under normal conditions (that is, with a free-falling nappe and without submergence), together with the turnout gate may be considered as a semimodule. Changes in water level in the delivery do not reflect in the measurement unless the level exceeds the height of the fixed crest. Any change in upstream level results in a change of discharge.

Adjustable length weirs are used in many installations to avoid errors in measurements introduced by small errors in observation of head when low flows are passing. By shortening the weir, thereby increasing the head for a given discharge, the percentage error is reduced.[15]

Devices That Provide an Equitable Basis for Charges.—A sixth functional class of measuring devices are those that do not, in themselves, control the flow but are operated in conjunction with separate controls and provide a totalized record of the flow volume that passes. They are effective over a considerable range of discharges and are not affected by changes in water levels upstream or downstream, provided the downstream level does not fall below a predetermined limit and the differential head lies between certain rather broad limits. The totalized record of the volume of flow, that may vary over considerable range of discharge, provides a direct, equitable basis for charges.

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[14] "Improving the Distribution of Water to Farmers by Use of the Parshall Measuring Flume," by R. L. Parshall, Colo. Agric. College Experiment Sta., Fort Collins, Colo., Bulletin 488, May, 1945.

[15] "Errors in Measurement of Irrigation Water," by Charles W. Thomas, Transactions, ASCE, Vol. 124, 1959, p. 319.

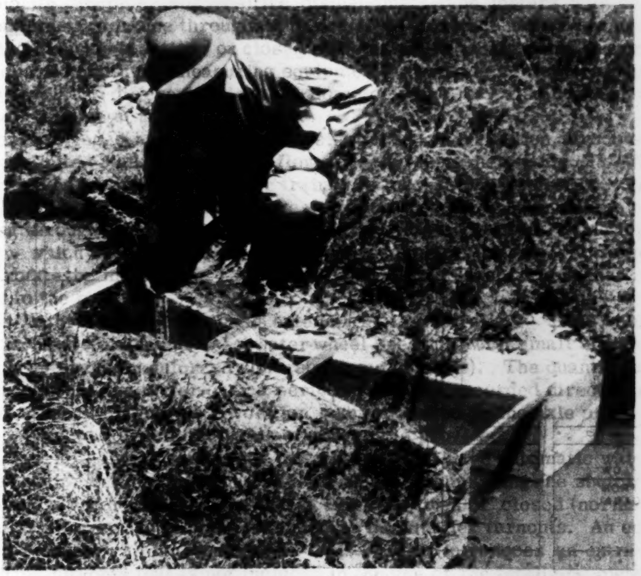

FIG. 21.—PARSHALL MEASURING FLUME

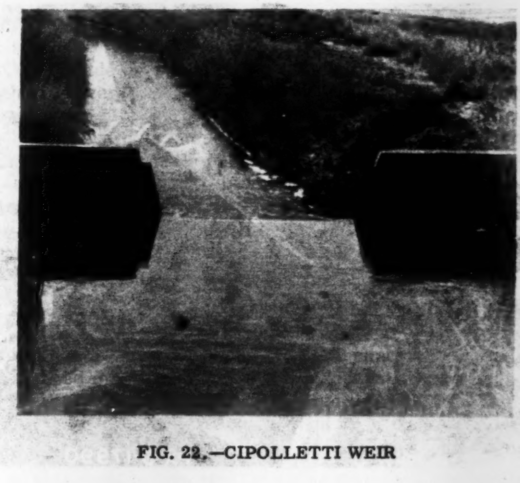

FIG. 22.—CIPOLLETTI WEIR

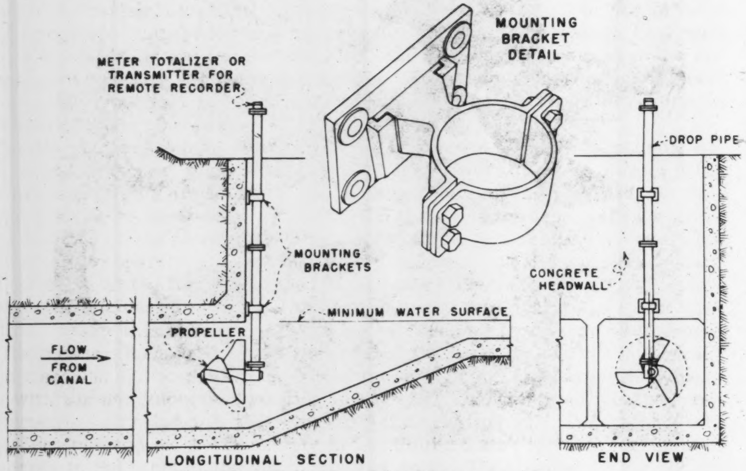
MOUNTING BRACKET DETAIL
METER TOTALIZER OR TRANSMITTER FOR REMOTE RECORDER
DROP PIPE
MOUNTING BRACKETS
CONCRETE HEADWALL
MINIMUM WATER SURFACE
PROPELLER
FLOW FROM CANAL
LONGITUDINAL SECTION
END VIEW

FIG. 23.—OPEN-FLOW METER ON FARM TURNOUT

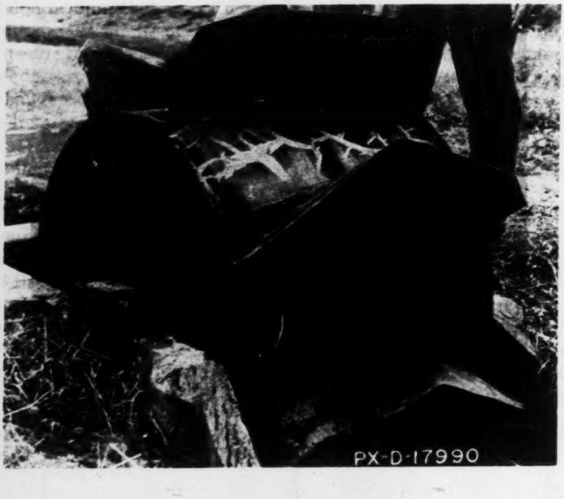

FIG. 24.—DETHRIDGE METER

Velocity, turbine, and positive displacement-type meters with totalizing mechanisms are in use throughout the world on closed conduit systems. There are also many installations on closed systems where instruments are employed in conjunction with the measuring equipment to provide a record of upstream water levels or differential levels. Some of these may also be found on open channel systems at the farm turnout.

A measuring device that is now being used on irrigation turnouts in the western United States is the open flow, velocity-type, meter, Fig. 23. The propeller drives an appropriate gear train with mechanical connection, to operate a totalizer which registers, in acre-feet or other units, the volume of water passing through the turnout irrespective of variations of head. Some irrigation systems which operate on a rotation basis also rotate the meters. Some operators rotate the meters, but have a totalizing unit for each turnout that remains locked to the turnout.

The Dethridge meter is used extensively in Australia.[16] This volumetric meter consists of an undershot water wheel operating with small clearances in a specially shaped reinforced concrete flume (Fig. 24). The quantity of water passing through the meter over a period of time is recorded directly in acre-feet by a specially geared revolution counter linked to the axle of the wheel. Thus, a direct and equitable basis of charges is provided.

In Algeria, some of the larger installations that have automatic water level control equipment and distributors use recorders employing the shutoff device to provide a record of the time that it is either open or closed (normal operation). This practice is being extended to the smaller turnouts. An equitable basis for charges is thus provided. This system introduces an extra step in computing charges that is not included in the use of the totalizing meters that register volume directly.

NEW DEVELOPMENTS

In the United States, the recent development of a vane meter shows some promise of success. The meter employs an elongated, roughly triangular-shaped vane. The sides of the triangle are curved toward the other and the vane is slightly convex in the direction of flow. The apex of the triangle is down and reaches into the flow. This meter is designed to indicate the rate of flow on a graduated scale as sensed by the vane in accord with the velocity and position of the water surface. In operation, the meter will be placed in a structure of rectangular cross section in which the water surface level need not remain constant. It is proposed that the instrument will be standardized, calibrated, and assembled to cover a range of flows adequate to meet the needs at farm turnouts.

CONCLUSIONS

The evolution of design of measurement and control devices for farm turnouts has progressed independently in widely separated geographic areas of the world. The result is an abundance of available designs. These may be classified into six general types. Each design has been developed to meet the demands of local conditions, and no doubt serves the specific needs in those areas. Many of the devices could serve as well in other areas. They all serve the same final purpose, namely, to control and measure the water to the user. However, to conclude that any one of the devices could be universally adopted would not be judicious.

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[16] "Water Measurement in Victorian Irrigation Districts—The Dethridge Meter," by I. Meacham, Annual Bulletin 1956, Internatl. Comm. on Irrig. and Drainage, p. 14.

Many of the devices cited are ingenious. Many utilize natural laws governing flow to accomplish the desired results. All have been produced through thought, work, and research, some more than others. Advantage should be taken of the opportunity to benefit from these developments. Integration of the desirable features of a number of the devices should result in improved present practices and might produce a solution that would be more universally applicable. Current extraordinary demands being placed on the available water resources of the globe may, in the near future, require this improvement.

DISCUSSION

LEE CHOW,[17] F. ASCE.—The author is to be complimented for this fairly complete review of the turnout structures for control and measurement of water. It is interesting to note the various types of turnout structures in use around the world. Although the author did not try to evaluate them, the discussions and future studies might bring out further information so that an evaluation could be made later on.

The turnout designs shown in Figs. 1, 2, 5, 6, and 12 are ingeneous. One drawback is there is no provision to shut off the flow when so required. Therefore, a simple perfection seems possible if gates are incorporated in these structures. The use of all the divisors shown in Figs. 7, 8, and 9 has, presumably, taken into account of the relative conveyance losses in the laterals due to their different lengths and the different soils they go through. That is, the discharges in the laterals are not strictly in proportion to the respective commanded areas.

The turnout structures, as the author mentioned, involve control and measurement of flows. In order to achieve this, either a combined structure or two separate structures are necessary. For the irrigation works in Taiwan, both types are in use. The combined types are the constant head orifices and the movable weirs. The separate types consist of a gate and some measuring device, either a standard weir, contracted or suppressed, or a Partial flume of free flow or submerged flow. All gates can be locked at any gate opening with a combination lock incorporated in the gate hoist. All these gates require some manipulation for the desired amount of flow. Once the operators are trained, they do not have any difficulty. Some experiences may be useful for the designers. Weirs and control gates have to be far enough apart to be free of turbulence of flow which makes reading difficult and inaccurate, but not unnecessarily far apart to cause inconvenience of operation. The air vents required for the suppressed weirs should be large enough to supply sufficient air under the nappe of the falling water. In some

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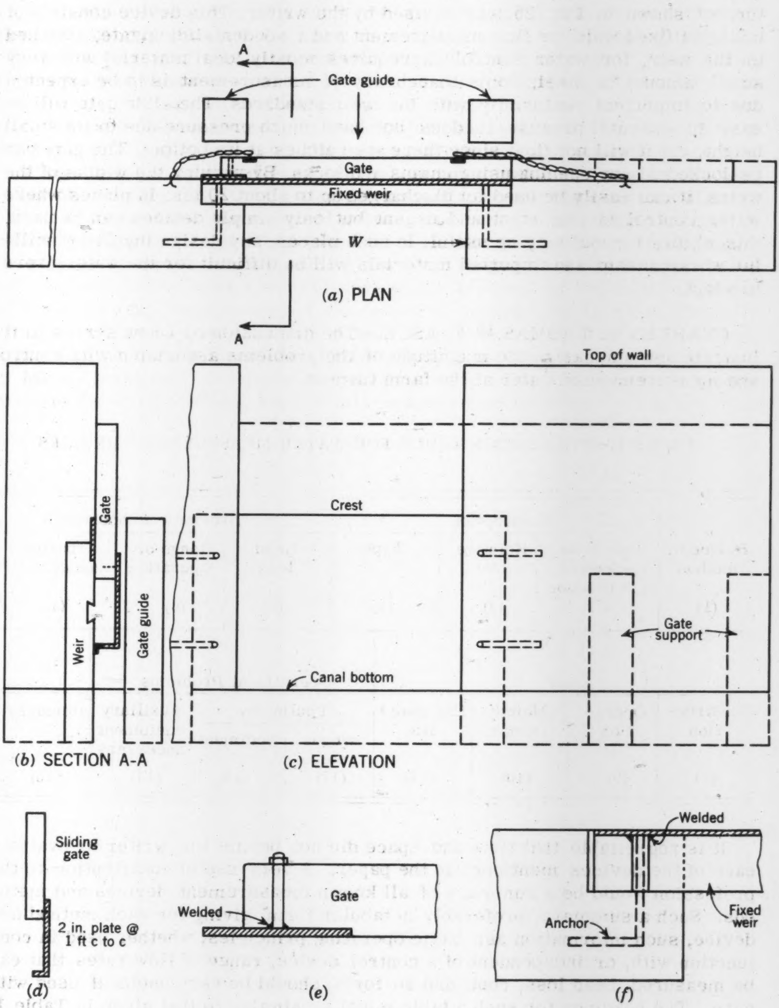
[17] Irrigation Engr., Food and Agriculture Organization of the United Nations, United Nations Technical Assistance Mission, Kabul, Afghanistan.


A
Gate guide
Gate
Fixed weir
W
A
Top of wall
Crest
Gate
Weir
Gate guide
Gate support
Canal bottom
Sliding gate
2 in. plate @ 1 ft c to c
Gate
Welded
Welded
Anchor
Fixed weir

(*a*) PLAN

(*b*) SECTION A-A

(*c*) ELEVATION

(*d*)

(*e*)

(*f*)

FIG. 25.—WOODEN SLIDE TURNOUT

canals, sediment may get behind the weir and affect the accuracy of measurement. In case of Partial flumes, proper selection of color and material for the staff in the stilling wells in necessary for easy reading.

In north Afghanistan, steel, iron and skill labor are often unavailable. In order to achieve some control and measurement of water, a wooden slide turnout shown in Fig. 25 was devised by the writer. This device consists of mainly a fixed weir for flow measurement and a wooden sliding gate, attached to the weir, for water control. It requires mostly local material with very small amount of steel. Some inaccuracy of measurement is to be expected due to imperfect conformity with the weir standards. The slide gate will be easy to operate because it does not have much pressure due to its small height, yet it will not float since there are catches at its bottom. The gate can be locked at any opening using chains and locks. By varying the widths of the weirs, it can easily be used for discharges up to about 20 cfs. In places where water control is important and urgent but only simple devices can be used, this sliding turnout may be useful. In such places, any design involving skillful workmanship and imported materials will be difficult for the water users to adapt.

CHARLES W. THOMAS,[18] F. ASCE.—The discussion by Chow serves to illustrate and emphasize the magnitude of the problems associated with control and measurement of water at the farm turnout.

TABLE 1.—EVALUATION TABLE FOR WATER MEASUREMENT DEVICES

	Description			Hydraulic Properties		
Device or method (1)	Operating principle and function (2)	Relative size (3)	Type (4)	Head loss (5)	Range of operation (6)	Expected accuracy (7)

Cost				Operational Problems			
				Fouling by			
Construction (8)	Operation (9)	Maintenance (10)	Estimated life (11)	Weeds etc. (12)	Sediment (13)	Auxiliary equipment necessary (14)	Remarks (15)

It is regrettable that time and space die not permit the writer to evaluate each of the devices mentioned in the paper. A very useful contribution to the profession would be a summary of all known measurement devices and methods. Such a summary, preferably in tabular form, giving for each method and device, such information as: basic operating principles, whether used in conjunction with, or independent of a control device, range of flow rates that can be measured, head loss, cost, and so forth, should be very useful if used with care. The headings for such a table might be similar to that given in Table 1.

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[18] Hydr. Engr., Bur. of Reclamation, U.S. Dept. of the Interior, Denver, Colo.

Perhaps this suggestion could be the stimulus for someone to expend time and effort to produce such an evaluation chart or charts.

It is particularly gratifying to the writer that Chow mentioned the desirability of having a single structure that will serve the dual function of control, regulation, and measurement of flows. As mentioned in the paper a shutoff is, in most instances, required at a farm delivery because of the manner in which the systems are operated. Economy in installation can be effected if the shutoff also serves as a regulator and as a means of measurement. Many times it may be economical or convenient to utilize control devices and structures as indications of the quantity of water flowing. Such structures are for the most part designed without thought of their use for this purpose. The difficulties of adapting them may be many, but the information resulting from calibration may be adequate for operational purposes and, in some instances, may be of a high order of accuracy. It must first be determined how well the control will serve for measurement and the expected accuracy of results. A calibration may be obtained by application of data derived from similar devices, by hydraulic model tests, or by measurements made in the field. Regardless of the method used to obtain a calibration of a control device, the accuracy of the calibration can be no better than the accuracy of the device or means used to effect the calibration. It may be seen then that caution is indicated in making such calibrations, and the results should be critically judged before general usage.

Many operational objections can be removed if a single device can be made to serve the two functions, control and measurement of flows.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
TRANSACTIONS

Paper No. 3274

ARTIFICIAL RECHARGE IN CALIFORNIA

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With Discussion by Messrs. Max Suter; and Raymond C. Richter
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SYNOPSIS

The author investigates some of the present needs for practice of artificial recharging of ground water in California. The major types of artificial-recharge projects used in California are described and the extent of artificial-recharge activities in California are summarized. Some of the important factors related to selection of artificial-recharge project sites are reviewed with particular emphasis on infiltration rates.

INTRODUCTION

California is richly endowed with extensive ground water reservoirs. Since the turn of the century, draft on ground water supplies in California has increased at an accelerated rate. In many areas, particularly in central and southern California, extractions of ground water are now far in excess of net natural recharge. Where these conditions exist, ground water is being withdrawn from storage, water levels are declining, and other adverse effects associated with ground-water overdraft are rapidly developing, including adverse salt balance and sea-water intrusion.

Note.—Published essentially as printed here, in December, 1959, in the Journal of the Irrigation and Drainage Division, as Proceedings Paper 2281. Positions and titles given are those in effect when the paper or discussion was approved for publication in Transactions.

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The primary purpose of most artificial recharge activities in California is to (1) combat adverse conditions resulting from overdevelopment of ground water, (2) increase conservation of local runoff, and (3) increase the amount of ground water available for use. Many specific objectives are encompassed by this purpose, including the following:

1. Augmentation of the recharge to the ground water reservoir to compensate for man's activities that tend to reduce natural recharge such as; (a) lining of stream channels for flood protection, (b) discharging of sewage and industrial wastes of suitable quality to saline waters through sewage-disposal systems, (c) sealing of natural-recharge areas with impervious sidewalks, streets, airports, parking lots, and buildings, and (d) exporting of local surface runoff that might otherwise percolate naturally in stream channels;
2. Conjunctive operation of surface and ground water reservoirs;
3. Reduction of ground water overdraft;
4. Control of sea-water intrusion and local bodies of poor quality water;
5. Control or prevention of adverse salt balance;
6. Maintenance or raising of water levels to avoid increased water-well construction, pumping costs, and obselescence of equipment and wells; and
7. Reduction or cessation of land subsidence problems in areas where overdraft results in compaction of sediments.

The term artificial recharge, as used in California, is defined as the process of replenishment of ground water storage through works provided primarily for that purpose. It should be noted that intent is an essential element in this definition. Many projects designed for disposal of return irrigation waters, sewage, cooling water, or other wastes will also augment ground waters. However, ground water replenishment is generally incidental to the primary function of these works, and they are not considered true artificial-recharge projects.

Numerous references related to artificial recharge were examined during the course of compiling basic data and preparing this paper. A partial bibliography of these references was prepared and is appended as general reference for the reader.

Some of the more important aspects of artificial recharge in California to be examined include; methods of artificial recharge, artificial-recharge activities in California, and factors related to selection of project sites with particular emphasis on infiltration rates.

METHODS OF ARTIFICIAL RECHARGE AND SOME DESIGN CRITERIA

Artificial replenishment of ground waters is accomplished primarily through works designed to maintain infiltration rates, increase the wetted area, and to increase the period of infiltration beyond that which exists under natural conditions. Although no two projects are identical, most of them utilize variations or combinations of the following basic methods: basins, modified stream bed, pits, ditches and furrows, flooding, and injection wells. These methods can all be used in conjunction with upstream surface reservoirs, that are sometimes provided to regulate flood flows, to temporarily store flood waters and limit flows to the absorptive capacity of the spreading works, and to remove a large portion of the detritus.

In most recharge projects, water is conveyed various distances from a source, such as an open channel or closed conduit, to the artificial-recharge project. Usually adequate facilities are provided for controlled diversions. In open channels, the diversion structures have ranged from a simple earth or rock dam thrown across a stream channel with little if any control of diversion, to a well designed gate-type installation that includes features to control the amount of water diverted and the time such diversion occurs. These structures are constructed so that they could be moved from a channel or destroyed during periods of extreme runoff to prevent obstruction of the channel that might cause overflow and cause subsequent flood damage. Flow measuring devices may be installed at the inlet structure or in the conduit to determine the inflow quantities and check the efficiency of the project. In many projects a conduit system similar to that previously described is provided to return waters that have passed through the project to the main stream.

A detailed examination of the seven principal methods of artificial recharge that are currently being utilized in California will be presented.

Basins.—By far the most common method of artificial recharge is through the use of basins. In the basin method, water is impounded in a series of basins that are formed by a network of artificial dikes or levees generally using to full advantage the surface contours. The basins are usually arranged in series, one above the other, so that overflow from the upper basin will flow into the next lower basin.

The basin method has many advantages in operation, including the following:

1. Basins utilize the maximum area for spreading with only the tops of the levees, that can be used for operation and maintenance, being unproductive. This is particularly important where suitable locations are scarce or land values are extremely high.
2. Irregular and gullied surfaces can be used with a minimum of preparation.
3. Silt-laden waters can be used, particularly if the upper basins are utilized for desilting and are periodically cleaned.
4. Considerable surface storage capacity is available in basins that can be used to store a portion of the water from flash floods for later slow percolation into the ground-water reservoir.
5. In general, local materials can be used for construction of dikes and levees.

The objective of plans for a basin-type spreading project is to obtain the maximum ratio of the wetted area to gross area commensurate with efficient operation and maintenance. In addition, smaller basins should be designed to obtain the highest ratio of the wetted perimeter to the wetted area. However, as the basin increases in size, the effect of shape diminishes rapidly. Basin size is largely dependent on the slope of the land surface; the flatter the slope, the larger the surface area of the basin for a given height of levee.

Levees can frequently be constructed by bulldozers, using the native soils without consideration to fill slope of compaction. However, in or near urban areas where seepage may damage private property, greater attention must be given to construction details. In general, the allowance for freeboard should range from 1 ft to 3 ft, depending on compaction. Roads are usually constructed on the main levees to facilitate patrolling, inspection, and operation and maintenance.

It has been found that greater flexibility of operation and maintenance can be obtained by having a minimum of two basins. This factor is important in continuous spreading projects because it permits the project to remain in operation during periods when spreading must be discontinued and basins dried for maintenance and re-establishment of infiltration rates. In projects designed to spread storm waters, multibasin operation has the advantage that the first of the series of basins can be utilized as a sedimentation pond to remove silt, which is among the more important maintenance problems. The upper pond should be large enough to substantially reduce the velocity of flow, and its inlet and outlet facilities should be so designed and located that short circuiting is prevented.

The design of any multibasin system should provide for adequate control of flow between basins. In most of the highly developed projects, the abutments of these interbasin control structures are made of concrete and the flow of water is controlled by standard size flashboards. There are many structures where abutments have been constructed of treated lumber, but under conditions of alternate wetting and drying it has been found that the wood generally deteriorates in a period of from 10 yr to 15 yr. Other installations, such as spillways and gated culverts, have been used with great success as control structures. In all cases, proper consideration must be given to erosive action on the downstream side of the interbasin structure caused by increased velocity, and, if necessary, riprapping should be provided to control this erosive action.

When this type of project is utilized to spread storm flow, a return structure should be placed at the low end of the project. This structure usually consists of a weir so placed as to direct the flow back into the stream.

Modified Stream Bed.—The utilization of stream channels represents an efficient and economical method of artificial recharge. Natural recharge from stream channels normally constitutes a large percentage of the natural recharge to a ground water reservoir. When the total flow in the stream channel exceeds the natural infiltration rate, the excess is lost by runoff. All efforts to increase percolation into the ground water reservoir are slanted toward conserving this excess runoff. Many agencies have constructed upstream reservoirs to regulate the erratic runoff and limit the flows to rates that do not exceed the absorptive capacity of downstream channels. Often, these stream channels are simply widened, leveled, or scarified to increase the infiltration.

In other areas small check dams and dikes are frequently used to reduce velocities and spread the flow over the entire width of the channel to effect maximum infiltration. These checks may be simply temporary earth structures protected by brush, wire, vegetative cover, riprap, and so on. In several streams, basin-like areas have been prepared by constructing temporary dikes of river bottom material. Temporary earth structures were built so as to quickly collapse when the flow in the river exceeds a given capacity. In some instances, this type of project has included permanent walls or weirs that have been constructed across river bottoms. However, this type of structure must be designed to take a battering from rocks, boulders, and other types of debris carried by the stream during periods of high runoff. Permanent structures are dictated by relative economics of initial cost versus maintenance and replacement expense.

Pits.—There are generally two types of pits. One type is the abandoned sand and gravel pit, that is very effective for recharging the ground water body after minor modification. Unless these unplanned pits have steep sides, relatively nonsilty water is used because of the difficulty in removing the silt from the sides as well as the bottom of the pit. The other type is the steep walled, planned "basin-like pit," that is deep with respect to its surface areal dimensions. The initial cost of both of these types of projects is generally low because all that is required is a conduit leading to the pit for the delivery of water. In both of these methods water is usually introduced into the pits by means of a pipe or open chute.

Artificial recharge by means of the planned "basin-like pits" is gaining in popularity. One of the reasons is the small initial capital cost of the project to the operating recharging agency. In many of the urbanized areas of southern California, sand and gravel operators will excavate this type of project to a given specification, in addition to which they will pay for the excavated sand and gravel. A second reason for the popularity of this method is its higher tolerance for silt than regular basins, because the silt usually settles rapidly to the bottom leaving the steep side walls relatively free for continued infiltration of water. For this type of project, provisions are usually made for rapid and economic removal of silt from the bottom of the pit.

Ditches and Furrows.—The ditch and furrow method consists of a system of flat-bottomed ditches that serve to transport water and provide percolation opportunity. This method can be used to supplement a basin spreading project. It can also be successfully applied to rough stoney terrain, and to areas in which slopes are too steep for basin construction. Waters with fairly high silt content can usually be satisfactorily spread, if velocities in the ditch systems are adequate to transport a large portion of the silt through the spreading system and deposit it in the main channel where it can be removed mechanically or carried out of the area during floods.

Several systems of orientation of furrows and ditches have been devised. In general, three basic types are commonly used: (1) contour type, in which the ditch follows the ground contour; (2) lateral type, in which water is diverted from the main canal to a series of small furrows; and (3) tree-shaped, in which the water is diverted from the main canal into successively smaller canals and ditches. The design size usually corresponds to the site size. It is generally restricted to sites in which land is relatively inexpensive, because the ratio of wetted area to gross area is usually very low, often averaging only about 10%.

The width of a ditch may vary greatly, but generally ranges from 1 ft to 6 ft, depending on the terrain and the velocity desired. It should be designed for velocities that will not erode the channels, but are high enough to carry the silt through the system. On very steep slopes, checks are frequently constructed for the two-fold purpose of minimizing erosion and increasing the wetted area.

In an endeavor to keep design, survey, and construction costs to a minimum, many agencies have first constructed a rough ditch by using only the simplest instrument, such as a plow. This simple facility is then improved by modification during initial operations. For this type of project, it is usually necessary to place a collecting ditch at the lower end of the site to return the water to the stream channel.

Flooding.—Flooding is a very effective and low cost method of recharge in areas in which slopes are gentle and uniform, and where gullies or ridges are small. In the flooding method, water is allowed to flow over the land surface in a relatively thin sheet. Generally it is best not to disturb the native vegetation and soil cover in the flooding area in order to maintain a favorable infiltration rate.

Ditches and weir-type embankments are usually used to initially distribute the water across the upstream end of the area to be flooded. Direction of flow is controlled by a few strategically placed embankments. Peripheral dikes are usually constructed to control the water sufficiently to prevent damage to adjacent property and to return water to the stream channel.

Injection Wells.—Injection of water into abandoned wells and wells specifically designed for artificial recharge has been practiced for many years with varying degrees of success. The use of injection wells is largely confined to those areas where surface spreading is not feasible due to the presence of extensive and thick impermeable clay layers overlying the principal water-bearing deposits. They may also be economically feasible in metropolitan areas in which land values are too high to utilize the more common basin, flooding, and ditch and furrow methods.

Many attempts to recharge ground water through injection wells have yielded disappointing results. Difficulties encountered in maintaining adequate recharge rates have been attributed to silting, bacterial and algae growths, air entrainment, rearrangement of soil particles, and deflocculation caused by reaction of high sodium water with soil particles. However, the Los Angeles County Flood Control District, in California, in cooperation with the California Department of Water Resources, has successfully operated injection wells as part of a large scale field experimental project concerning the feasibility of creating and maintaining a fresh water ridge to halt sea water intrusion in the Manhattan-Redondo Beach area in Los Angeles County. Favorable percolation rates have been maintained by chlorination and deaeration of the water supply, and by conducting a comprehensive well maintenance program.

The spacing of the injection wells depends on the range of influence of a well, that is in turn dependent on the amount of water to be recharged through the well, and the acceptance rate of the aquifer. The acceptance rate is a function of the aquifer permeability, hydraulic gradient, the length of casing penetrating the aquifer, and the number of casing perforations.

In general, it has been found that gravel packed wells operate more efficiently and require less maintenance than do nongravel packed wells. At Manhattan Beach, Calif., a 24-in. gravel packed well with an 8-in. casing was found most desirable for recharging purposed. In addition, where water is being injected under pressure, it has been found that a concrete seal should be provided on the outside of the casing at a point at which it passes through the relatively impermeable cap, to prevent the upward movement of water along the outside edge of the casing.

With respect to perforations, consideration should be given to using a "Moss perforator," which makes horizontal louvered slits in the casing. This type of perforation is particularly advantageous in a predominantly sandy formation because it inhibits movement of sand from the formation into the well when injection pressure is relieved. In addition, it is suggested that the casing perforations be placed below the normal water table to lessen the chances of chemical incrustation.

A well-header assembly must be provided to bring the water to the recharge well and to regulate the flow of water into the well. In general, the water to be recharged should be maintained at a relatively constant pressure, and should not be allowed to fall freely in the well, because the resulting aeration greatly affects acceptance rates. The design of this assembly will vary greatly, depending on the purpose for which the project will be used.

It should be emphasized that if a long term project is contemplated, treatment of water is imperative. Sediment must be almost completely removed and the clear water should be treated with relatively high concentration of chlorine, calcium hypochlorite, or copper sulfate to prevent the formation of bacterial slime and algae. In addition, care should be taken to insure that no water containing a high percentage of sodium is injected, because such water will cause deflocculation of the aquifer sediments and rapid decrease in transmissibility of the aquifer. This problem is most prevalent in soils containing clay and silt, and not very important in soils made up primarily of sand and gravel.

ARTIFICIAL RECHARGE ACTIVITIES IN CALIFORNIA

Artificial replenishment of ground water reservoirs in California has been practiced for many years. As early as 1895, flood waters of San Antonio Creek in southern California were conserved by spreading on the alluvial fan at the mouth of San Antonio Canyon. The practice of artificial recharge has increased to such proportions that it is presently a major method of conservation of water in California.

The California Department of Water Resources is conducting a comprehensive investigation and inventory of all artificial-recharge activities in California. The inventory indicates that artificial recharge activities are concentrated in three areas: (1) Santa Clara Valley, south of San Francisco Bay; (2) the central and southern part of the San Joaquin Valley; and (3) southern California. This is readily understandable, in light of the extensive exploitation and overdevelopment in these areas, coupled with extreme variability of runoff, scarcity of suitable surface storage, and adverse water quality problems such as sea water intrusion, salt balance, and related problems.

This inventory will be presented by examining the agencies conducting spreading, type and numbers of recharge projects, and quantities of water recharged.

Agencies Conducting Spreading Operations.—As of October 1, 1958, there were fifty-four agencies actively practicing artificial recharge in California. The majority of the operating agencies conducting artificial recharge are public agencies such as irrigation districts, water conservation districts, and flood control districts. A summary of the type and number of agencies having active projects in California as of October 1, 1958, is as follows:

Type of Agency	Number of Agencies
Irrigation Districts	9
Water Conservation Districts	7
Flood Control Districts	7
Other Water Districts	7
Municipalities	4
Mutual Water Companies	10
Other Water Companies and Associations	6
Miscellaneous	4
TOTAL	54

Projects by Methods of Spreading.—As of October 1, 1958, there were 276 active artificial recharge projects in California. All of the standard methods of placing water underground (modified stream bed, flooding, basins, pits, ditches and furrows, and injection wells) are represented in this listing. The basin method is the most commonly used method, constituting 149 projects or 54% of all projects. Other methods include modified stream bed with 15%, ditches and furrows with 8%, pits with 7%, flooding with 4%, and injection wells with 12%.

Quantities Recharged.—The total quantity of water recharged to all ground water basins in California is not known with any degree of accuracy. However, the best estimate based on reported amounts artificially recharged during the period 1912-13 through 1957-58, on the basis of only the recharge facilities for which reports were obtained, totaled about 6.25 million acre-ft. Of this amount, about 800,000 acre-ft were imported from the Mono Basin-Owens Valley and the Colorado River and were spread in coastal southern California. The highest annual volume reported during this period, 630,000 acre-ft, was attained in 1957-58, and this water was reported as being spread in sixty-three projects in California. Of the total amount spread in 1957-58, about 170,000 acre-ft was Colorado River water reported spread in coastal southern California.

TABLE 1.—QUANTITIES OF WATER SPREAD

Method (1)	1957-58 Acre-feet (2)	Per cent (3)	1912-13 through 1957-58 Acre-feet (4)	Per cent (5)
Basin	367,700	58.4	2,790,900	44.7
Modified Stream Bed	185,900	29.5	2,340,500	37.5
Ditch & Furrow	59,400	9.4	1,050,100	16.8
Pit	8,500	1.3	18,400	0.3
Well	6,100	1.0	28,600	0.5
Flooding	2,400	0.4	12,500	0.2
TOTAL	630,000	100%	6,241,000	100%

By far the largest quantity of water has been spread by the basin method with the least quantity spread by the flooding method. Table 1 indicates the reported quantities of water spread by various methods in the period 1957-58, and 1912-13 through 1957-58.

Cost.—The cost of artificial recharge projects varies greatly with such factors as (1) purpose, (2) method of spreading, (3) quantity, quality and regiment of flow of water, (4) the surface and subsurface conditions, (5) the location of the artificial recharge project, and (6) the standards and requirements of the agencies. The large difference in cost of a project is exemplified by two basin type projects, both of which were constructed in about 1955. A relatively simple basin type project containing 30 acres of gross area and 26 acres of wetted area, was constructed on an alluvial fan in an unpopulated area at a total cost of less than $10,000 or considerably less than $500 per acre of wetted area of development. In contrast, a very sophisticated basin type project, containing 125 acres of gross area and 110 acres of wetted area, was constructed in a flood plain area and highly productive citrus area at a cost of about $580,000, or more than $5,000 per acre of wetted area. It is evident that there is no meaningful comparison between costs of projects in California. However, it may be of value to examine some of the factors associated with and affecting the total cost of projects in California.

The cost of land has sometimes been a large item in the total cost of projects in California. It varies mainly with location and the time at which the purchase was effected. For example, the present worth of land at many projects located in the populated areas of southern California is over $3,000 an acre as compared to their original cost of about $200 an acre in 1930.

Also, natural resources have often been found on lands purchased for artificial recharge projects. These resources have lessened the net costs of projects. An example is the sand and gravel found on project sites located in the populated areas in coastal, southern California. Sand and gravel operators have bought the material as well as removed the material under specified conditions for planned development of a basin or pit type project.

In general, lands were acquired when the need arose, but in the past, some agencies, following a master plan, have bought large acreages whenever extra funds were available. In these cases, inexpensive methods of spreading, such as the ditch and furrow type, and flooding type of operation, were immediately started to get some benefit from the land. Further improvements were made from time to time as other funds became available. Some lands were sold or even traded for more favorably located land. In some cases, legal and court fees were involved when public agencies revert to condemnation procedures.

Standards of requirements for engineering, construction, and operation and maintenance differed between agencies, that in turn, affected the cost of spreading projects. The standards and requirements were generally dictated by the type of development in and around the area, being highest in fully developed urban areas. For example, in urban areas, the cost of fencing and of patrolling the project area has added significant costs to the spreading operation.

Another item that has affected the cost of spreading projects is the type and size of appurtenances, such as installation for diversion, the conduits for conveying the water from source to the spreading project, and the installation to return the water from the spreading project to the main stem. When costs of these appurtenances were large in proportion to the development of the spreading ground itself, the cost per unit area of one project was considerably higher when compared with other projects in which cost of appurtenances was not a major item.

A study was made to relate the cost of operation and maintenance to a unit amount of water spread. Operation and maintenance cost could be classified as either fixed cost or variable cost. The fixed cost portion of operation and maintenance, such as rent, utilities, taxes, and insurance, for spreading a unit volume of water varied with the amount spread, being greater when a smaller amount of water is spread. Thus, this fixed cost for spreading a unit volume of water is expected to be less during wet periods when compared with dry periods. However, the variable cost portion of operation and maintenance, such as cost of silt removal, operating personnel, and patrolling, may be great, even though a large amount of water was spread due to the inefficient use of personnel and spreading grounds. In general, high efficiency is reached whenever the spreading project is operated at design capacity. For example, about 5,000 acre-ft of local storm water and about 22,000 acre-ft of imported water were spread in a project in southern California during 1957-58. Because the release of imported water was scheduled for desired periods and controlled to the design capacity of the spreading ground, the cost of operation and maintenance in spreading a unit volume of imported water was less than when spreading a unit volume of local storm water.

FACTORS RELATING TO SELECTION OF ARTIFICIAL RECHARGE PROJECT SITES

Numerous factors enter into the selection of a potential recharge site. The engineer and geologist must carefully evaluate these factors, individually and collectively. Some of these factors include the following:

1. availability and character of local and imported water supply;
2. factors affecting infiltration rates, such as natural ground slope, surface soils, and geologic and subsurface hydrologic conditions;
3. operation and maintenance problems;
4. net benefits;
5. water quality considerations; and
6. legal considerations.

However, this discussion will be confined to the first four items listed, because the latter two items will be presented in other papers.

Availability and Character of Local and Imported Supply.—The first and probably most important factor to consider is the availability of local or imported water. The design, operation, and efficiency of a recharge installation are influenced to a large degree by the source, nature, and regimen of the available water supply. Thus, consideration should be given to evaluation of two different types of flow; the perennial supply in which the discharge rate is relatively constant, and the more erratic supply in which the discharge rate changes rapidly.

Where a perennial supply of water is available, operation can be continuous and a high degree of efficiency can be attained. This condition can be realized where surface storage is utilized in conjunction with ground water reservoirs or where underground storage is utilized for regulation of imported water supplies.

The direct use of surface runoff may present difficult design and maintenance problems, particularly in the arid west where stream flow is erratic, and floods are violent. In order to effect any appreciable conservation of flood flow, provision must be made for handling large volumes of water in short periods of time. Loss of absorptive capacity through deposition of silt must be controlled, either by rejection of initial heavily-laden flows, temporary detention in surface storage, or maintenance of velocities adequate to carry silt through the recharge works without deleterious settlement. However, water supplies can be utilized under extremely adverse conditions by proper design of installations and adequate maintenance of facilities.

Factors Affecting Infiltration Rates.—There are many factors affecting infiltration rates, most of which are difficult to separate in order to determine their magnitude and effects. All the various items that make up the surface soils, and geologic and subsurface hydrologic conditions directly affect infiltration rates. The quality of the water and the procedure used in the construction, operation, and maintenance of projects also greatly affect infiltration rates, although the use of these items can be somewhat controlled to maintain desirable infiltration rates. In this presentation natural ground slope, surface soils, and geologic and subsurface hydrologic conditions will be examined to illustrate some of the factors affecting infiltration rates. Also presented will be information on using these items as guides in obtaining an estimate of the infiltration rate for a project site.

Because infiltration rates vary with time, the long time infiltration rate was selected to study the effects of natural ground slope, surface soils, and physiographic positions. The long time infiltration rate is defined, for use in this paper, as that rate that exists after a period of spreading from two to four weeks, depending on the character of the surface and subsurface conditions. In this definition it was assumed that the rate of infiltration will not be affected by the ground water mound beneath the spreading site. Long time infiltration rates were obtained for about one-hundred artificial-recharge projects. The majority of these projects are located in southern California on alluvial fans and flood plains on which the soil textures are predominantly fine-to-stoney sands. Long time infiltration rates were also obtained for projects located in the San Joaquin Valley on low lying alluviated areas composed of soils that have predominantly silty to fine sand loam textures. Some of the soils in the San Joaquin Valley are characterized by clay pan or iron hardpan layers that impede infiltration rates.

For this study, the principal physiographic features on which artificial-recharge projects are located were selected as basis of relating long time infiltration rates to the various subsurface conditions, because a detailed determination of subsurface conditions is generally complex, time consuming, and expensive. The following are brief definitions of these principal features:

1. Alluvial fan is a fan or cone shaped topographic feature that has been deposited by one stream or river.
2. Alluvial fan stream channel is that portion of an alluvial fan which contains a recent, or active stream channel. It also includes that portion of the fan where a stream channel would have existed if the stream were not controlled by works of man.
3. Flood or coastal plain is the broad, relatively flat, gently sloping topographic feature that has been formed and controlled by a through flowing stream or river. The coastal plain is further characterized by a depositional area that is essentially controlled by sea level and is terminated by the coast line.
4. Inland alluviated valley is mainly confined to the San Joaquin Valley which is characterized by low alluvial fans and plains.

Natural Ground Slope.—The natural ground slope is somewhat related to the surface soils and the subsurface geologic conditions. In some instances it may be difficult and expensive to obtain data on surface soils and subsurface geologic conditions, especially the latter. Because it is usually easy and inexpensive to secure the natural ground slope from topographic maps, natural ground slopes may be conveniently used as a guide in obtaining an estimate of long time infiltration rate for a project site.

In California, the majority of the artificial recharge projects are located in the slope range of 10% to 0.1%. As shown in Fig. 1, the long time infiltration rate increases with increasing slope. The line of best fit to all the points showing the relationship between long time infiltration rate in feet per day and natural ground slope in per cent, is expressed by the equation $I = 2.14 + 1.85$

Log S. This equation shows that for a slope (S) of 10%, the long time infiltration rate (I) was about 4 ft per day. Similar values for slopes of 1% and 0.1% are 2.1 ft per day and 0.3 ft per day, respectively. The line that describes the upper limit of this relationship is described as

$$I = 4.39 + 3.26 \text{ Log } S \ldots\ldots\ldots\ldots\ldots \text{(1a)}$$

The equation of the line describing the lower limit of this relationship is

$$I = 0.67 + 0.78 \log S \ldots\ldots\ldots\ldots\ldots \text{(1b)}$$

Surface Soils.—Soil types are an important factor in the establishment of initial infiltration rates. In general, the coarser the texture of the soil, the higher are the initial and sustained infiltration rate. There are other chemical, physical, and organic properties of a soil that affect infiltration rates.

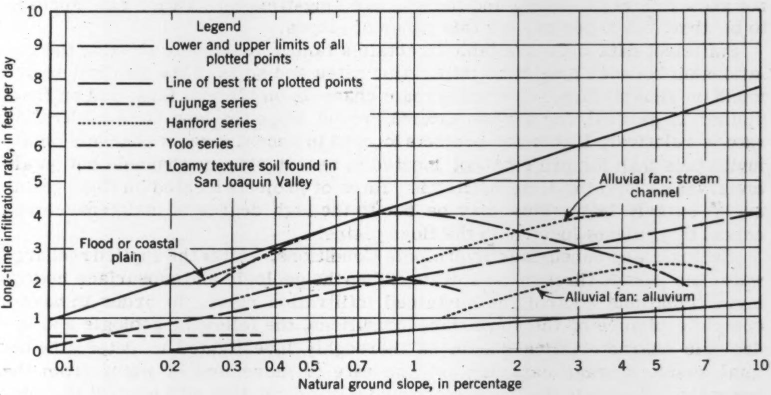

FIG. 1.—RELATIONSHIP BETWEEN INFILTRATION RATES AND NATURAL GROUND SLOPES FOR VARIOUS SOIL TYPES

These properties include (1) permeability, (2) claypan and iron or lime hardpan development in the soil profile, (3) depth of soil profile, (4) organic matter, (5) type of crop culture, and (6) degree of compaction in construction, operation, and maintenance of the grounds.

Soil type may be inexpensively identified by using existing soil maps. Where soil maps are not available, a quick reconnaissance can be accomplished by strategic placement of several hand auger holes to determine the soil profile. In addition to the natural ground slope, soil type may be conveniently used as a supplemental guide in obtaining an estimate of infiltration rate for a project site.

Existing soil maps were utilized for determining the major soil types at each recharge project in California. The predominant soils are the Tujunga, Hanford, and Yolo series and the loamy texture soil types existing in the San

Joaquin Valley. All soil types found at spreading projects, with the exception of the San Joaquin Valley soils, are permeable, have good surface and subsurface drainage, are generally lacking a claypan or hardpan in the soil profile, and are low in organic matter.

As shown in Fig. 1, the long time infiltration rate for a given series generally increases with increasing slope until a maximum value is reached. Then the long time infiltration rate decreases with increasing natural ground slope. In general, the highest infiltration rates occur in those recharge projects underlain by the soils of the Tujunga series. A maximum infiltration rate of 4 ft per day on this soil appears to occur with a natural ground slope of 0.9%. This is probably accounted for by the uniform grain size of the soil near the 0.9% slope in which the ratio of voids to total mass of this type of soil is at a maximum. A similar relationship is noted in the Yolo series, with a maximum infiltration rate occurring at 0.75% natural ground slope. The fine loamy soils in the San Joaquin Valley in the natural ground slopes of about 0.1% to 0.3% do not show this relationship, and the average long-time infiltration rate appears to be about 0.5 ft per day for this range of slopes.

Sufficient data were available to obtain a family of curves showing the relationship between long-time infiltration rates and soils of the Hanford series found on alluvial fans, in recent stream channels on alluvial fans, and on flood plains. In general, for a given natural ground slope the long-time infiltration rate is relatively higher for projects located in recent stream channels on alluvial fans than for projects not located in recent stream channels but on alluvial fans. The long-time infiltration rates of projects located on flood plains are relatively high. This may be due to the high degree of maintenance received by projects located on the flood plains.

Geologic and Subsurface Hydrologic Conditions.—After the applied recharge water has passed through the soil zone, then the geologic and subsurface hydrologic conditions control the sustained infiltration rates. In order to have a complete picture of the subsurface conditions, the following geologic and hydrologic characteristics should be thoroughly investigated to determine the total usable storage capacity, and the rate of movement of water from the spreading grounds to the areas of ground water draft, that will control the sustained infiltration rate:

1. physical character and permeability of subsurface deposits, and depth to ground water;
2. specific yield, thickness of the deposits, and position and allowable fluctuation of the water table;
3. transmissibility, hydraulic gradients of the water table and pattern of pumping; and
4. structural and lithologic barriers to both vertical and lateral movement of ground water.

A detailed determination of subsurface conditions is generally time consuming and expensive. However, physiographic positions that are generally related to many of these subsurface conditions can be easily and inexpensively determined. In addition to natural ground slope and soil type, physiographic position may be conveniently used as another supplemental guide in obtaining an estimate of long time infiltration rate for a project site.

Artificial recharge projects in California are generally located on the four physiographic positions (1) alluvial fan, (2) alluvial fan—recent stream channel, (3) low alluvial fan—San Joaquin Valley, and (4) flood or coastal plain. As shown on Fig. 2, the long-time infiltration rates of projects located in a given physiographic position generally increase with increasing natural ground slope until a maximum value is reached. Then it decreases with increasing natural ground slope. The highest long-time infiltration rates appear to occur in the alluvial fan—recent stream channel at about 3.5% natural ground slope. As the slope increases above 3.5%, the long-time infiltration rate decreases. A similar relationship is found in the alluvial fans where the maximum infiltration rates occur at about 4.5% natural ground slope. Infiltration rates for the flood or coastal plain areas increase with increasing slope, and the maximum rate appears to occur at about 0.85% natural ground slope. Average long-time infiltration rates in the San Joaquin Valley remain relatively constant at about 0.5 ft per day for natural ground slopes ranging between 0.1% and 0.3%.

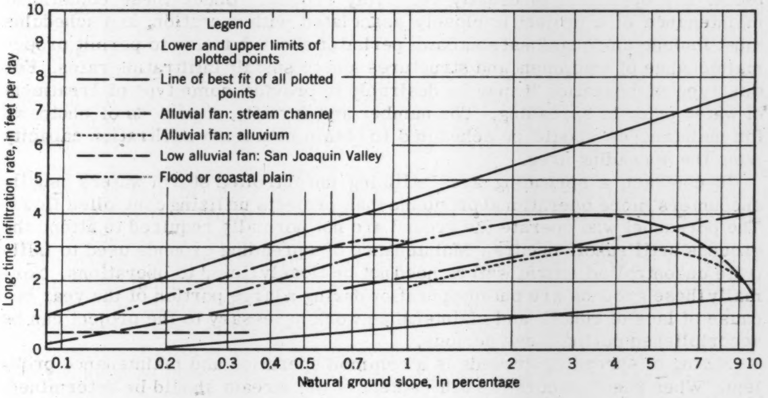

FIG. 2.—RELATIONSHIP BETWEEN INFILTRATION RATIO AND NATURAL GROUND SLOPES FOR VARIOUS PHYSIOGRAPHIC POSITIONS

A study was made to determine the relationship between long-time infiltration rates and specific yield values of materials underlying the spreading grounds. The specific yield values were based on interpretation of well logs. Although long time infiltration rates were expected to increase with increasing values of specific yield, no relationship was found. This may have resulted from the large variation in the interpretation of subsurface material by well drillers.

Operation and Maintenance.—It is axiomatic that any project should be operated and maintained to obtain the maximum benefits for which the project was specifically designed. The details involved in the operation and maintenance of a specific artificial recharge project vary greatly with the purpose, the method used, the character of the water, and the location of the project. However, there are a number of basic problems that can be anticipated. These problems not only affect infiltration rates, but also affect such important conditions as public health and safety. Some of these problems, their manifestations, and corrective actions which may be taken are set forth in Table 2.

In order for the project operator to do an efficient job, he must know the purpose of his operation, and the details and capacities of the physical features of the project. In addition, he should know the actions to be initiated in case of emergencies or failures. It has been found that specific operating instructions have been extremely helpful, particularly when the project is large and complex, and when spreading is periodic as in the conservation of storm runoff. Such an instruction sheet should include among other matters the purpose of the project, a clear definition of the duties and responsibilities of the operator, instructions relating to items to be measured and operation of measuring devices, and instructions on the use of special equipment.

In an area in which spreading is contemplated on a continuous basis, personnel are usually assigned to operate the grounds on a more or less permanent basis, and operation is usually relatively routine. Under these conditions, maintenance of a project is closely associated with operation, and schedules must include an occasional shutdown period of short duration to permit proper maintenance of equipment and structures and to sustain infiltration rates. For this type of operation it may be desirable to provide some type of treatment of water prior to spreading. The number and duration of periods of shutdown for maintenance should be scheduled to obtain maximum infiltration capacity from the spreading area.

In contrast, a spreading area utilizing uncontrolled storm waters usually encounters more operational problems than projects utilizing controlled flows. The personnel who operate the ground are not normally required to attend the grounds until runoff occurs. Maintenance of spreading grounds used to infiltrate uncontrolled storm waters need not be closely timed to operations. Normally these grounds are out of operation during a large portion of the year because of lack of runoff, and maintenance work necessary to the project can be accomplished during these periods.

Silting of spreading grounds is a common operation and maintenance problem. When runoff occurs the silt content of the stream should be determined. When the silt content is reduced to what is estimated by experience to be an allowable concentration, diversion can be initiated. The allowable silt content in the water diverted for spreading varies from project to project with the method of spreading, vegetative cover, surface soil, and relative cost of removing the silt from the spreading grounds. The allowable concentration of silt at each project is usually determined by experience. It is usually less than 1,000 ppm. Generally 10,000 ppm is the maximum limit. One agency does not permit the concentration of suspended and precipitable solids to exceed 20 ppm in water diverted for spreading.

In diverting to spreading grounds of uncontrolled flood water, care must be taken to fill the spreading grounds slowly to prevent breakdown of uncompacted dikes and other earthfills. In addition, constant patrol must be maintained to observe sloughing, leaking, erosion, or other signs of structural failure.

Evaluation of Net Benefits.—Consideration of any plan to artificially replenish a ground water body presupposes the desirability or necessity of augmenting the existing water supply or the use of the ground water body for storage and distribution of local and imported supplies. An evaluation of net benefits is necessary to enable the engineer to decide whether or not to conduct artificial-recharge operations. An evaluation of net benefits of a surface water system can be measured directly as the amount of revenue obtained as a result of the project. However, when dealing with artificial replenishment, the amount of benefits obtained may not be easily related to tangible form. In many cases, the organization conducting the operation will realize, at best, only partial direct benefit. Furthermore, the problem may be complicated by factors of ground water hydrology and quality that may be exceedingly difficult to evaluate accurately in monetary terms.

In general, the benefits to be derived through artificial replenishment of ground water basins may be broadly grouped into two categories. These are (1) relief of overdraft on the ground water basin, and (2) use of ground water basins as reservoirs and distribution systems.

Relief of Overdraft.—There are certain computable benefits that are immediately apparent where artificial replenishment is conducted for the purpose of relieving overdraft on a ground water basin. These include a possible decrease of energy charges for pumping as a result of a reduction of pumping lifts, the prevention of possible capital expenditures for deepening of wells and the lowering of pumps, and the prevention of possible premature abandonment of wells.

Benefits that would be difficult to compute can be derived where replenishment of an overdrawn basin can prevent sea-water intrusion, the release of deep seated connate brines, or the possible dewatering of the basin or portions thereof. Any one of these reactions could result in the partial or complete failure of the underground basin to yield a continued supply of water. Some measure of this benefit might be derived from a determination of the cost of replacing the lost facility with an equivalent surface system. However, the value of the ground water basin as an emergency supply is inestimable.

To obtain maximum benefits from artificial recharge however, the ground water basin must have the necessary geologic and hydrologic characteristics to permit the infiltration and transmission of the water required to relieve the overdraft. If these characteristics are not adequate for the purpose, some combination of ground water basin development and surface water distribution system must be developed. In this case, the net benefit of recharge would be evaluated as a difference between the cost of a surface system designed to completely supply the area under consideration with water from an available water supply and comparable costs of a system to supplement the ground water supply with water from the same available water supply.

Use of Ground Water Basin as Reservoir and Distribution System.—The benefits of utilizing a ground water basin as a reservoir for the storage and regulation of surface supplies can be measured by the saving in cost of equivalent surface storage reservoirs and appurtenant facilities. This saving will be particularly great at locations where there are few, if any, natural reservoir sites and surface regulating facilities may be exceedingly expensive.

In order to derive benefit from the use of a ground water basin as a reservoir, geologic and hydrologic conditions must be favorable for the desired storage and regulation. The availability and development of ground water storage capacity to meet projected requirements for regulation of both local water and imported supplies, as well as the sufficient transmissibility in the aquifers to permit the movement of the spread water from the point of replenishment to the point of extraction must be considered. The usable capacity of the ground water reservoir can be developed by planned extractions of the ground water during

TABLE 2.—OPERATION AND MAINTENANCE PROBLEMS

Problem	Manifestation	Corrective Actions to be Considered
Silt	Lodging of particles within interstices of soil near the surface area, reducing the infiltration rate.	(1) Desilt in retention reservoir and/or in uppermost series of basins. Flocculating agent such as "Separan" has been used with success. (2) Bypass water until concentration of silt will not be detrimental, with concentration depending upon soil condition. Ditches and furrows generally can accept waters containing higher concentrations of silt if sufficient velocity is maintained through the project to carry silt back to the main canal. (3) Scrape, harrow, and/or disc after proper drying. Period of drying usually ranges from one to seven days depending upon soil and weather conditions. (4) Remove silt after drying. Silt may be used to build up levees of basins or bridges of ditches or furrows. (5) Sustain vegetative growth. (6) Sluice the silt out of ditches and furrows, and from channels, with due regard to erosion problems. (7) Pump injection well to loosen silt from interstices and remove silt from the well.
Weeds	Increases percolation rate and shortens drying period required for working an area or removing silt from the basin. There is a disadvantage in that vegetative growth may be a fire hazard.	(1) Control by chemical means and/or remove when weeds become a fire hazard, especially around structures. Consider use of hand labor instead of mechanical means in order to maintain infiltration rates. However, if possible, leave vegetation undisturbed in wetted area. (2) Prolonged deep submergence will kill vegetation. (3) The control of weeds is generally not considered a problem in the operation of pits and shafts, or injection wells.
Rodents	Leaks and failures of dikes and levees. Public nuisance near urban area.	(1) Set out poison about twice a year. (2) Use of traps.
Public health and safety	Rodents and mosquito problem and possible injury to individuals. Potential problems of injury is greatest when depth of water is large in basins and pits.	(1) Enclose area with fence and gates with locks. (2) Patrol area with particular attention to children and structural failures, before and during operation, especially near inhabited area. (3) Vector control by use of mosquito fish, chemicals, and/or drying. (4) Rodent control by poisoning or traps. (5) Proper posting of signs when using chemical which is poisonous.

Maintaining of percolation rates	Reduction of percolation rates will decrease efficiency of system, increasing the unit cost of the amount of water actually recharged.	(1) Proper treatment of water. Desilt water to concentration desired. Use chlorine or copper sulfate for control of bacterial slime and algae. Use of chemicals to reduce the possibility of chemical incrustation, which is usually deposition of calcium carbonate. (2) Schedule intermittent drying periods to prevent problems due to swelling of soil particles. Permit growth of vegetation to decrease the drying period by removing the water in the root zone and loosening the soil. Studies have shown that bermuda grass has been successfully used to maintain rates, even under prolonged periods of deep submergence. (3) Prevent aeration of water, especially when operating recharged wells, pits, and shafts. (4) Increase head of water generally by increasing depth of water. (5) Use hand labor whenever possible to decrease the possibility of using heavy equipment which will cause surface compaction especially when soil is wet. (6) Scrape, harrow, and/or disc after proper drying. (7) Remove silt, chemical incrustation, and/or any material decreasing infiltration rates after proper drying period. (8) Maintain the design velocity to reduce silting to a minimum in use of ditches and furrows. (9) Recondition injection wells by use of dry ice, hydrochloric acid, and/or sulfuric acid. (10) Prevent freezing of water during winter by continuous spreading. (11) Check the possibility of base exchange reactions. (12) Soil can be reconditioned by using organic material such as cotton gin trash, or or chemical agents, such as krilium.
Maintenance to diversion structures	Breakdown of spreading operations.	(1) Systematic and routine maintenance check as well as patrolling when in operation. Attention should be given to wooden structures since they deteriorate faster due to frequent wetting and drying cycles. Also attention should be given to settlement of structures thus changing flow condition. (2) Attention should be given to under-cutting of structure particularly on the downstream end which preventive maintenance primarily in the form of riprapping. (3) Sluicing of channel to remove silt and debris which have accumulated near and at diversion structure.

periods of deficient supply and subsequent replenishment during periods of surplus surface supply in much the same manner as a surface reservoir would be operated.

One of the largest benefits to be derived is the saving in cost of developing equivalent usable capacity in a surface storage reservoir. A second large but computable benefit when using the aquifer as a distribution system is the savings derived in terms of the difference of cost of a surface distribution system to supply part or all of the demands with due regards to peak requirements. An additional benefit of the utilization of a ground water basin is the saving in water that would be lost by evaporation from a surface reservoir. This could be computed as the cost of the water saved. An aspect that is very difficult to evaluate is the high degree of protection from contamination that is characteristic of ground water. This immunity, together with elimination from danger of destruction of reservoir structures and the wide dispersion of outlet facilities that can be attained, makes the ground water basin of value as an emergency supply, particularly in the event of nuclear warfare.

One of the more important negative benefits is the excess loss of water due to consumptive use of water by phreatophytes in areas in which high ground water tables would exist. Encroachment on storage required to conserve local runoff in order to regulate imported supplies must be considered a negative benefit.

IMPORTANCE OF ARTIFICIAL RECHARGE OPERATIONS TO WATER CONSERVATION ACTIVITIES IN CALIFORNIA

The California Department of Water Resources has developed a comprehensive master plan for the control, protection, conservation, distribution, and utilization of the waters of California. This plan, known as The California Water Plan, has been formulated to meet the present and estimated future needs of the state to the maximum feasible extent. The role of artificial recharge plays an important part in this over-all plan.

Incorporated in this plan is a system of major facilities to redistribute excess waters from northern areas of surplus to areas of deficiency throughout the state. It is estimated that the total regulatory and conservation storage required under the plan exceeds 100 million acre-ft, more than 80 million acre-ft in excess of existing surface storage facilities. In the Central Valley alone, 40 million acre-ft of additional storage are required, far beyond the capabilities of remaining suitable surface sites. The solution to this problem lies in large scale utilization of underground reservoirs that, fortunately, are readily available. This plan presently contemplates a combination of 22 million acre-ft of surface storage in a series of foothill reservoirs and 31 million acre-ft of underground storage for meeting ultimate water requirements in the Central Valley. The operation of these surface- and ground-water reservoirs would be coordinated to achieve optimum conservation. This would be accomplished by drawing on ground water during dry periods, thus creating storage space in underground reservoirs to be utilized for storing excess runoff during ensuing wet years. Lands would be supplied alternatively from surface or ground water storage, as dictated by prevailing conditions. Replenishment of ground water reservoirs would be accomplished by artificially recharging local and imported waters through facilities provided for this purpose, by percolation of normal excesses of applied irrigation water and of precipitation during wetter years, and by seepage of water from surface distribution canals, most of which would be unlined to increase percolation opportunity.

Although this proposed "conjunctive operation" of surface and ground water reservoirs is unprecedented in scope and magnitude, there are actually no new principles involved, and there is every confidence that it constitutes the most feasible method for fully developing the water resources of the state. It is recognized, however, that many legal, financial, and operational problems must be solved before the plan can be implemented. Utilization of both surface- and ground-water resources must be coordinated on a regional or basin wide basis; existing water rights must be established and recognized; and costs must be apportioned on an equitable basis. However, under the urgencies generated by the tremendous growth of the state's economy, solutions will be found for all these problems, and the practicability of "conjunctive operation" of surface and ground water storage on a grand scale will be fully demonstrated.

CONCLUSIONS

1. Artificial recharge of ground water reservoirs has been one of the important conservation practices in California. From 1912-13 through 1957-58, approximately 6.25 million acre-ft of water was reported to be artificially recharged. Most of the recharging activities are located in Southern California.
2. Artificial recharge will continue to play an increasingly important role in the optimum development of water resources in California through the combined use of surface and ground water reservoirs. Under this "conjunctive operation", artificial recharge will be instrumental in relieving overdraft conditions in many ground water basins in California, and will allow planned utilization of ground water basins as reservoirs and transmission systems to economically meet fluctuating and growing water demands.
3. Six principal methods of artificial recharge utilized in California include: basins, modified streambeds, pits, ditches and furrows, flooding, and injection wells.
4. The basin method is the most commonly used method, constituting 149 projects or 54% of all projects in use in 1958.
5. The majority of operating agencies conducting artificial recharge are public agencies such as irrigation districts, water conservation districts, and flood control districts.
6. Infiltration rate is one of many significant factors that must be investigated in the selection of an artificial recharge site. Natural ground slope, surface soils, and geologic and subsurface hydrologic conditions are key factors that must be studied in detail in order to arrive at a close estimate of infiltration rate at a site.
7. When limited time and funds preclude detail study of the many factors affecting infiltration rates, the natural ground slope, soil type, and physiographic position can be used as guides to approximate infiltration rates for potential sites.
8. Long-time infiltration rates of artificial recharge projects in California generally range from 0.5 ft per day to 4.0 ft per day. Long-time infiltration rate curves correlating natural ground slope, soil types, and physiographic positions have been developed, based on long-time infiltration rates in 100 arti-

ficial recharge projects in California. (Correlations are presented in Figs. 1 and 2).

9. Major operational problems in California artificial recharge projects are silt, weeds, rodents, public health and safety, maintaining of percolation rates, and maintenance to diversion structures.

ACKNOWLEDGMENTS

Grateful acknowledgment is made to the many individuals who have contributed to the information presented herein. The writers are indebted to the following coworkers in the California Department of Water Resources who assisted in compiling basic data on artificial recharge projects in California: Edward Whisman, Senior Engineer Water Resources; John Anderson, Assistant Civil Engineer; Sanford L. Werner, Assistant Engineering Geologist; and William Jones, Junior Engineering Geologist. Appreciation is also due Max Bookman, M. ASCE, District Engineer; Lucian Meyers, M. ASCE, Principal Hydraulic Engineer; Donald H. McKillop, M. ASCE, Senior Engineer Water Resources; and Robert G. Thomas, Senior Engineering Geologist, who reviewed and commented on the original draft of this paper.

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64. "Present Economic and Technical Status of Water Reclamation from Sewage and Industrial Wastes," Univ. of California, Berkeley, Calif., San. Engrg. Research Project, Technical Bulletin No. 4, March, 1951.

65. "Summary Report Pertaining to Basic Parameter of Sea-Water Intrusion and the Hydraulics of Injection Wells," Univ. of California, Berkeley, Calif. San. Engrg. Research Project, May 25, 1953.

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67. "Progress Report - Study of Problems Associated With Ground Water Recharge Barrier to Prevent Chloride Infiltration," Univ. of California, Berkeley, Calif., Dept. of Engineering, December, 1952.

68. "A Report Summarizing Progress on Hyperion Waste Water Reclamation Research Project," by H. A. Van der Goot and R. M. Hertel, Los Angeles Flood Control District, March 30, 1956.

69. "Maintenance and Operating Problems of Water Spreading Grounds in Southern California," by K. G. Volk, Transactions, Amer. Geophysical Union, Vol. 15, Part II, 1934.

DISCUSSION

MAX SUTER,[3] F. ASCE.—This discussion deals only with two phases of the paper, namely recharge rates and bibliography.

Recharge Rates.—The amount of recharge has increased greatly, especially since Colorado River water is available. To get this water into the ground, more agricultural land is used in recharge areas with a system that allows rates of infiltration seldom over 4 ft per day. No efforts seem to be made to increase this rate, although success in such an undertaking would allow additional increase in recharge without the use of additional farm land. The paper indicates that the reason for the lack of search for an improved method lies in a satisfaction with the present results and a shortage of understanding of the hydraulic conditions on which recharge is based.

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This fact can be recognized from the discussion of the influences affecting infiltration rates. Not considered have been such factors as ground water level below spreading site, the depth of the ground water stream, the slope of the bottom formation of the ground water stream, the ability of the ground above it to absorb the recharge water and to spread it over a larger area, whose extent and volume should be known. These factors are important, as any recharge operation can only continue for a long time and to large amounts when the recharge water has a place to go. And naturally the easier the water can flow from the pit, the higher the rate of recharge.

Recharge is on an area, the wetted area, whereas spreading is on the circumference in linear order, the area increases faster than the circumference and, therefore, for larger areas, the inflow can be greater than the outflow.

Considering these conditions, the Illinois State Water Survey was able to reach recharge rates of 60 ft, 100 ft and even up to 200 ft per day; that is, 15 to 50 times the maximum rates obtained in the California operations. The Peoria operation has been fully described elsewhere.[4,5]

Bibliography.—Ordinarily the bibliography gives data concerning works cited in the text or lists the main works containing additional important information. Neither of these two cases is applicable to the bibliography attached to the paper. There is not a single reference to any publication in the text. The grade of importance of the papers listed will not be judged here, but it is remarked that the list is incomplete relative to some very important papers.

Completeness of a bibliography probably can never be attained, even if only American literature is listed. If foreign literature is included, (and in scientific references this should always be the case), completeness is more difficult, even if limitations are placed on date and content. With clear definitions, the latter may still be subject to personal judgment and thereby be out of a "satisfying everybody" class.

RAYMOND C. RICHTER,[6] Aff. ASCE, and ROBERT Y. D. CHUN,[7] M. ASCE.—It is agreed that there is a definite need for improved techniques to obtain and maintain high long-time infiltration rates. As Suter is probably aware, there are many agencies currently engaged in studies to establish procedures for increasing the rate of placing water into ground water basins. The University of California, United States Soil Conservation Service, and the Texas High Plains Underground Water Conservation District are examples of agencies conducting studies of this type.

Suter lists some geological and hydrological factors that are important to any discussion of influences affecting infiltration rates. These and other factors have been presented in the paper.

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[4] "The Peoria Recharge Pit: Its Development and Results," by Max Suter, Proc. Paper No. 1102, ASCE, Vol. 82, November, 1956.

[5] "High-Rate Recharge of Ground Water by Infiltration," by Max Suter, Journal, AWWA, Vol. 48, April, 1956, p. 355.

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